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TYPES AND DETAILS OF BRIDGE CONSTRUCTION

PART III. SPECIFICATIONS AND STANDARDS FOR SHORT RAILROAD SPANS

Recent practice of great railroads, leading bridge shops and eminent bridge engineers. Exemplified by abstracts, tabulation and comparisons of many notable specifications and illustrations and descriptions of standard details and typical designs. With discussions by eminent railroad bridge engineers and an appendix on elevated railroad superstructures

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Classified and tabulated for facility of reference and comparison and for advantageous use in designing

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PREFACE.

Specifications and Standards Short Railroad Spans.

The great increase in traffic and train loads has necessitated the reconstruction within the last few years of a large proportion of short-span bridges on the leading railroads in this country, and the abolition of grade crossings in most large cities and towns has greatly increased the number of new structures required.

Adaptability to high speeds, rigidity, freedom from vibration, simplicity and economy of construction, ease of maintenance, endurance under overloading, resistance to train accidents, facility and rapidity of erection and replacement are such important factors of plate girder spans that they have become generally adopted for openings of about 15 to 100 ft., and thus include a very large proportion of short-span R.R. bridges.

The perfection of steel manufacture, the development of bridge-shop tools of great accuracy and large capacity, the doubling of the capacities of freight cars and the use of powerful derricks, derrick cars, improved hoisting engines and large hydraulic jacks have greatly encouraged and facilitated the construction, shipping and erection of girders of dimensions previously impracticable, and have made comparatively long plate girder spans common.

Their desirability has been recognized by many progressive railroad-bridge engineers and special consideration has been given to their design; standard types have been adopted; loading, clearances, principal dimensions and proportions have been determined; details best adapted to shop equipment, good workmanship, commercial materials, and efficient service have been developed and their construction has been so far perfected that few changes in details are now being made by prominent designers who until quite recently have made frequent radical variations in their work. The maximum dimensions of plate girder

spans are now limited by the possibility of railroad shipment, and their weight per linear ft. is as much greater than that of truss spans of the same length as is considered justifiable by their superior advantages. So that it is not likely that present practice will be materially changed unless by some radical development in materials, as might perhaps result from the improvement and cheapening of nickel steel if it were great enough to cause it to displace the present quality of structural steel.

The specifications are the fundamental element of design in structural work, and these have been very widely revised and perfected by all bridge engineers within the last decade. No part of the ordinary specification has been universally accepted by them, but many clauses possess a considerable degree of uniformity, a few are agreed on by groups of designers, (many of whom, however, have simply accepted the dictums of notable specifications, although some have arrived at nearly or quite the same result independently), many have minor differences, vary between limits, or apply the same principles to different assumptions, and many others go to opposite extremes and present the widest divergence in requirements and conclusions for similar cases.

The ground has been so thoroughly and laboriously covered that for most ordinary purposes the designer, unless an expert of unusual ability and opportunity, or where exceptional conditions are involved, can secure more valuable results from a wise selection and suitable adaptation or composition of existing data than by attempting to originate new formulæ, requirements and details. It is therefore the object to gather together here as many as possible of the latest and most important specifications, arranged and classified so as to make them most available for reference and comparison, and to facilitate the selection and combination of portions suited to the specific requirements of any given design or investigation.

Requirements which are substantially uniform in a large number of specifications are arranged in one group, and those which vary greatly are in most cases enumerated in another group. All the requirements which can be briefly expressed are tabulated for comparison and arranged so that their variations may be readily noted. Special requirements of different specifica-

tions are excerpted to indicate the essential variations from general practice and complete the presentation of all the essentials of all the specifications without numerous repetitions of matter essentially the same.

Standard details are hardly separable from the specifications and in a number of cases have been adopted by the railroads to insure uniformity of design. Separate detached details, applicable to different spans, and drawings of complete spans which are typical of the standards and general character of the work required by certain specifications are both illustrated in Part II.

The floor construction in short-span bridges differs markedly from the girder construction in that it has not yet attained general uniformity except in cases where the open type, with the cross ties supported directly on deck girders or on stringers, is adopted. In short, through spans the grade must often be kept as low as possible and the thickness of the floor reduced to the utmost, thus necessitating very shallow floors of comparatively great weight. These, and floors designed to carry ballast and maintain the track uniform with that at the ends of the bridge, have been made with many forms of solid deck and beam construction, and a growing tendency is apparent to adopt a combination of steel and concrete which gives a large mass and provides efficient protection for the metal with small maintenance expense. The principal types of such floors are illustrated in Part III., where the development can be traced from early to recent construction.

The considerations which should govern the design and construction of short spans, the methods employed and the special features considered most important are ably presented in Part V. by discussions contributed by prominent railroad bridge engineers to whom the author makes acknowledgement for their valuable contributions.

It was at first intended to include the contents of this volume in Part II. of Types and Details of Bridge Construction, but as it proved difficult to restrict the data to the amount allowable for a single volume, it was divided on the line of recognized standards and it is believed that the two volumes, while entirely complete and independent separately, together fully cover the construction of short-span steel bridges in this country and

afford valuable data, conveniently classified and arranged to illustrate advanced practice, facilitate the investigation and study of standard requirements and construction, permit comparisons between them and aid in the selection of the most efficient or suitable details for a given structure.

Most of the illustrations have been made from special drawings prepared from official blueprints and shop drawings in which no changes have been made except in the elimination of unnecessary or immaterial data so as to emphasize the essential features and avoid confusion, thus illustrating the character of the design without giving the minute details necessary for working plans. The descriptions of standard and typical designs have been intended to supplement the illustrations and present such essential features as are not covered by the corresponding specifications or included in the requirements of ordinary shop and office practice. In some cases they have been copied from articles prepared by the author for the pages of The Engineering Record and the others have been written especially for this book from data furnished by the designers of the structures.

The 35 specifications which have been abstracted and tabulated have been selected from a much larger number, by a process of elimination by which those of least importance, incomplete ones, essential duplicates and especially those that have not been recently revised were omitted. It is regretted that some important specifications requested for this purpose were not available and that some others which are here included have not been revised to conform with all the latest improvements actually adopted by their authors in current work, but such discrepancies are in most if not all cases insignificant. The specifications under consideration include those of many of the principal railroads, bridge engineers, and bridge shops, and contain most essentials of other specifications not included. All of them have been submitted to the writers or present engineers for approval and revision and the corrections made substantially bring them up to their latest practice. Sincere thanks are due to the authors of these specifications for their valuable assistance in this work.

FRANK W. SKINNER.

TOMPKINSVILLE, N. Y., 1906.

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PART I

SPECIFICATIONS

BRIDGE CONSTRUCTION

PART I

SPECIFICATIONS

Bridge specifications should serve as a basis for estimates and contracts and be authority for disputed points in design, material, and workmanship. They should secure uniformity, excellence, and the latest improvements in construction; provide standards for the principal features of design and execution, and control the final decision of important questions of strength, detail, manufacture, and points in dispute between the parties of the contract. They should define general conditions and requirements, explicitly state what points are fixed and what may be adopted in accordance with the contractor's convenience and equipment when approved by the engineer. They should contain all rules and formulæ necessary to the complete calculations for normal structures, and give all assumed data needed, together with necessary and explanatory diagrams. They may well be accompanied with tables of stresses, values, and estimates computed from the data given or assumed, and by plans and details suited to be used with varying dimensions for regular work.

Specifications should help secure the greatest strength, safety, durability, and simplicity of structures, together with the least ultimate total cost, considering repairs and maintenance. They should make the designs conform to the best standards of high grade shop practice, and provide for reasonable future increase of duty, at the same time encouraging a conservative advance in the quality of materials and workmanship and improvement of details. In this way they can eliminate many faults of careless or poor designs, and, with competent inspection, insure that the structures are built as contracted for and are of good quality. In capable hands they will minimize unintelligent work, and secure the best and most economical results, but they should

never be considered to suffice without experienced designers and inspectors.

The inspectors should be able engineers with special shop and mill training, and fully informed concerning the conditions relative to the work. They should use the specifications as a basis of contract with the builders, and have power to deviate from their letter, to increase or relax their severity in special cases to expedite or improve the work as well as to relieve the contractor from severe hardship when it can be done without serious detriment to the work.

Specifications should vary according to the range and class of structures they are intended to govern and the interests and people by whom they will be used; thus those used in the office of a bridge engineer specialist and by able designers may evidently be much briefer and more elastic than those intended for promiscuous use or for engineers in general practice; a different class of specifications will be inevitably issued by bridge shops and rolling mills desiring to sell products adapted to their special facilities and interests, and special specifications adapted to the conditions should be prepared by a bridge engineer for important or unusual structures for which competitive designs and bids are invited.

Steel structural specifications did not exist thirty years ago, and the earlier steel bridge specifications, based on those for wrought and cast iron structures, have been radically changed within the last fifteen years. A much greater uniformity of methods of calculation, assumed data, essential requirements and general features of design has been since attained, and rigidity, strength, massiveness, and durability have been considerably increased to conform with the greater weight and speed of live loads, increased train service, improved quality of steel, and great development of bridge-shop machine tools and rolling-mill products.

Most of the principal railroad and bridge engineers and bridge builders have revised their specifications within the last few years; from these and from some others, in most cases not more than five years old, the following collation has been prepared for easy reference and comparison, and includes such worthy specifications as were readily available for the purpose, most of the important ones not included being those which were not furnished when requested for this purpose, or those which were then in process of revision and were therefore not available for publication. This presentation is not properly a standard speci-

fication, nor does it give complete or uniform data, but it is of interest and value in that it shows the limits of present practice and indicates most of the important or unusual features of design and construction.

Excepting the well-known elementary principles of mathematical and graphical determinations of stresses, nearly all data for the calculations, detailing and construction of plate girder work are here included and all important points covered by the references cited. These data are presented impartially; some of them are conflicting, and some divergences are probably due to special conditions not here apparent; their presentation does not imply approval or verification, they are simply given as facts. In most cases the original specifications are not quoted verbatim, and unimportant clauses are uniformly omitted, but care has been taken to abstract the essential paragraphs so as to give their important meaning briefly and clearly and as far as possible with a nomenclature uniform with other similar paragraphs, sometimes retaining the original wording, but oftener revised, abbreviated, and rearranged.

CHAPTER I

UNIFORM REQUIREMENTS

A LARGE majority of the authorities here represented, agree substantially on the following points, although in some cases they are evidently considered so much a matter of course that they are not mentioned.

Effective Length and Depth used in calculations shall be respectively, center to center of end bearings, and distance between centers of gravity of flanges.

Calculations must provide for heavy locomotive excess, impact, wind pressure, centrifugal force, and brake action.

Steel must contain limited percentages of sulphur, phosphorous, and carbon, must have ultimate tensile strength between upper and lower limits, elastic limit between limits or proportional to ultimate strength, minimum elongation and reduction of area at point of fracture, must be subject to cold bending and drifting tests, and must be extra soft and ductile with limited ultimate strength, for rivets.

Net Sections only, must be considered in proportioning tension members; the diameter plus $\frac{1}{8}$ in. must be deducted from the width of a member for each rivet hole in its cross-section.

Shearing Stresses must be resisted wholly by the web.

Bending Moments must be resisted wholly by the flanges, except where a portion of the web is assumed to act with the flanges.

Thickness of Materials must not be less than $\frac{3}{8}$ in. except for fillers.

Web Plates must be cut scant at aplices and on top and bottom edges, and must be spliced with a pair of cover plates having at least two rows of rivets each side of the joint.

Flanges must generally be of T-shape cross-section made up with a pair of angles back to back, horizontal legs outwards, and one or more cover plates, and must have same gross areas top and bottom. Side plates and additional pairs of angles are sometimes used to secure extra heavy sections.

Flange Angles must form at least half the total flange area, or must be of the largest size made. They must be riveted to

the web with rivets proportioned to transfer the shear at any point in a length equal to the depth of the girder.

Flange Cover Plates must not extend more than a limited distance beyond flange angles; must extend beyond their theoretical length; must generally be spliced by overlapping ends of outer plates, and must be stayed at given points.

Flange Joints must be milled; must be staggered on opposite sides of the web; must be as few as possible, and must have the fewest possible field-driven rivets.

Web Stiffeners usually consist of single angles back to back on opposite sides of the web, except where two angles on the same side of the web are used for floorbeam connections of through spans. They are usually fitted closely at the ends, and are of uniform size throughout unless larger at the ends. They are located at bearings and points of application of concentrated loads, and intermediately at distances not greater than the depth of the girder.

Lateral Bracing of riveted angles is provided in the planes of the loaded flanges.

Vertical Transverse Bracing usually consists of diagonal angles between top and bottom, horizontal angles at the ends and at intermediate points of deck spans.

Gusset Plate kneebraces are usually used at the ends of floorbeams in through spans.

Rounded Upper Corners are generally required at the shore ends of through spans, the end vertical angles usually having the same width as the top flange angles, and being continued around the curve and spliced to the top flange angles on tangents.

Sliding End Bearings are used for the expansion ends of short spans, and have their contact surfaces planed, and are secured against transverse displacement.

The following requirements for drilling, punching, riveting, reaming, painting, and inspection are quoted from the American Bridge Company's specifications, and substantially represent the general practice.

Punching. All riveted work shall be punched accurately with holes $\frac{1}{16}$ in. larger than the size of the rivet, and when the pieces forming one built member are put together, the holes must be truly opposite; no drifting to distort the metal will be allowed; if the hole must be enlarged to admit the rivet, it must be reamed.

Drilling. All holes for field rivets, excepting those in con-

nections for lateral and sway bracing, shall be accurately drilled to an iron templet or reamed while the connecting parts are temporarily put together.

Reaming. In medium steel over $\frac{5}{8}$ in. thick, all sheared edges shall be planed, and all holes shall be drilled or reamed to a diameter of $\frac{1}{8}$ in. larger than the punched holes, so as to remove all the sheared surface of the metal.

Pitch. The pitch of rivets, in the direction of the strain, shall never exceed 6 in., nor 16 times the thickness of the thinnest outside plate connected, and not more than forty times that thickness at right angles to the strain.

Spacing. The distance from the edge of any piece to the center of a rivet hole must not be less than one and one half times the diameter of the rivet, nor exceed eight times the thickness of the plate; and the distance between centers of rivet holes shall not be less than three diameters of the rivet.

Riveting. The rivet heads must be of approved hemispherical shape, and of a uniform size for the same size rivets throughout the work. They must be full and neatly finished throughout the work and concentric with the rivet hole.

All rivets when driven must completely fill the holes, the heads be in full contact with the surface, or countersunk when so required.

Wherever possible, all rivets shall be machine driven.

Bolting. When members are connected by bolts which transmit shearing strains, the holes must be reamed parallel and the bolts turned to a driving fit.

Painting. All iron work before leaving the shop shall be thoroughly cleaned from all loose scale and rust, and be given one good coating of pure boiled linseed oil, well worked into all joints and open spaces. In riveted work, the surfaces coming in contact shall each be painted before being riveted together. Pieces which are not accessible for painting after erection shall have two coats of paint. All surfaces in contact shall be painted before they are put together. After the structure is erected, the iron work shall be thoroughly and evenly painted with two additional coats of paint, mixed with pure linseed oil, of such quality and color as may be selected. Finished surfaces shall be coated with white lead and tallow before being shipped from the shop.

Inspection. All facilities for inspection of material and workmanship shall be furnished by the contractor to competent in-

spectors, and the engineer and his inspectors shall be allowed free access to any part of the works in which any portion of the material is made. The contractor shall furnish, without charge, such specimens (prepared) of the several kinds of material to be used as may be required to determine their character.

CHAPTER II

VARYING REQUIREMENTS

In the following principal points many variations, sometimes amounting to contradictory requirements, prevail in different specifications as indicated by the succeeding tabulations and extracts: Limiting lengths of girder spans, Proportionate depths of girders, Spacing of girders, Test requirements, Rivet values, Allowance for field riveting, Live loads, Unit values of dead loads, Location of girders on curves, Proportionate sections of middle and outer girders, Types of bearings, Lengths of girders without provision for expansion, Amount of expansion provided for, Variation of temperature provided for, Pressure on masonry, Use of fillers under pedestals and bed plates, Proportioning bed plates, Proportioning rollers in expansion bearings, Maximum thickness of materials, Spacing web stiffeners, Maximum dimensions, Pitch and gauging of rivets, Tension in rivets, Locking nuts on bolts, Reaming with templates, Direction of fiber in web plates, Straightening of materials, Clearances, Handling materials, Loading and shipping girders, Connection of lateral diagonals to stringers, Anchorages, Elevation of outer girder on curves, Bearings of track ties on deck girder flanges, Squaring skew ends, Permissible variations of weight, Unit stresses proportioned to variations of span and of dimensions of materials, Analytical and graphical methods of calculating shear and moments, Allowance for impact, Camber, Deflection, Vibration, Speed and static tests of finished structures, Records, Preliminary data, Uniformity of scale and drawings.

In the following tables, notes, and extracts, specifications are referred to simply by the reference numbers here prefixed.

1. American Railway Engineering and Maintenance of Way Association, 1906.
2. Atchison, Topeka and Santa Fe Railway System, 1902.
   James Dun, chief engineer; A. P. Robinson, bridge engineer.
3. Baltimore and Ohio R. R. Company, 1904.
   D. D. Carothers, chief engineer; J. E. Greiner, engineer of bridges and buildings.
4. Choctaw, Oklahoma and Gulf Railroad, 1902.
   Boller and Hodge, consulting engineers.

5. Boston Elevated Railway Company, 1907.
G. A. Kimball, chief engineer; R. B. Davis, bridge engineer.

6. Boston and Maine Railroad, 1896.
J. P. Snow, bridge engineer.

7. Canadian Pacific Railway, 1903.
C. N. Monsarrat, engineer of bridges.

8. Chicago Great Western Railway, 1904.
A. Münster, bridge engineer.

9. Interborough Rapid Transit Company, 1902.
S. L. F. Deyo, chief engineer.

10. Standard Specifications, Association American Steel Manufacturers.

11. Cleveland, Cincinnati, Chicago and St. Louis Railway, 1901.
G. W. Kitteredge, chief engineer.

12. General Specifications for Steel Railroad Bridges and Viaducts, 1901.
Theodore Cooper, consulting engineer.

13. Illinois Central Railroad Company, 1904.
A. S. Baldwin, chief engineer; H. W. Parkhurst, engineer of bridges and buildings.

14. The King Bridge Company, 1898.
H. W. King, president; W. P. Brown, chief engineer.

15. New York Central and Hudson River Railroad, Leased and Operated Lines, 1904.
W. J. Wilgus, chief engineer; Olaf Hoff, engineer of structures.

16. Railway Bridge Superstructures.
The Osborn Engineering Company, 1903.
Wilbur J. Watson, bridge engineer.

17. Phœnix Bridge Company, 1895.
John Sterling Deans, chief engineer.

18. Market Street Elevated Passenger Railway Company, Philadelphia Rapid Transit Company, Lessee, 1904.
W. S. Twining, chief engineer; C. M. Mills, engineer of bridges.

19. Philadelphia and Reading Railway Company, 1904.
W. Hunter, chief engineer.

20. Board of Rapid Transit Railroad Commissioners, New York, 1902.
W. B. Parsons, chief engineer.

21. Southern Railway Company, 1902.
D. W. Lum, general chief engineer maintenance of way.

22. Union Pacific Railroad Company, 1904.
J. B. Berry, chief engineer.

23. Pennsylvania Railroad Company, 1905.
A. C. Shand, chief engineer; H. R. Leonard, engineer of bridges and buildings.

24. New York, Chicago and St. Louis Railroad, 1904.
E. E. Hart, chief engineer; A. J. Himes, engineer of bridges.

25. General Specifications for Steel Railroad Bridges, 1906.
Virgil G. Bogue, C. E., by Albert W. Buel, associate on bridges.

26. Western Maryland Railroad Company, 1906.
J. Q. Barlow, chief engineer; V. G. Bogue, consulting and supervising engineer; A. W. Buel, associate on bridges.

28. Western Pacific Railway Company, 1906.
V. G. Bogue, vice president and chief engineer; A. W. Buel and W. A. Aycrigz, bridge engineers.

29. Vera Cruz and Pacific Railroad Company, 1899.
Waddell and Hedrick, consulting engineers.

30. Pennsylvania Lines West of Pittsburg, 1906.
J. C. Bland, engineer of bridges.

31. Erie Railroad Company, 1900.
Mason R. Strong, engineer of bridges and buildings.

32. Michigan Central Railroad Company, 1904.
B. Douglas, bridge engineer.

33. Lake Shore and Michigan Southern Railway, 1904.
Samuel Rockwell, assistant chief engineer; R. H. Reid, general bridge foreman.

34. New York, New Haven and Hartford Railroad, 1901.
C. M. Ingersoll, Jr., chief engineer; W. H. Moore, engineer of bridges.

35. Wabash Railroad Company, 1905.
A. O. Cunningham, chief engineer.

36. Chicago, Milwaukee and St. Paul Railway, 1904
C. F. Loweth, bridge engineer.

37. Long Island Railroad Company, 1905.
J. R. Savage, chief engineer; J. B. French, bridge engineer.

CHAPTER III

PROPORTIONS, MATERIALS, LOADS AND STRESSES

Length of Span. The length of span for which plate girder construction is recommended or required.

TABLE 1.

Specification Numbers.	Span in Feet.		Qualifications.
	Shortest.	Longest.	
11	18	100	
11	100	125	Plate or lattice girders.
4–37–25–26–28–17.	20	100	
12	20	75	
12	75	120	Plate or lattice girders.
18	20	80	
21	20	105	
32–35	20	90	
22	19	100	
33	23	100	
15	25	100	
2	26	106	
8	28	80	Through spans.
8	28	100	Deck spans.
3	30	110	
30	0	100	
24	0	100	
31			
13	19	100	Up to 40 ft. long, no field rivets.

TABLE 2. — DEPTH d OF PLATE GIRDERS IN TERMS OF SPAN S.

Specification Numbers.	Value of d.
8–12–15–35	$\frac{1}{10}\ S$
3	$\frac{1}{12}\ S$, maximum 10 ft.
11–25–26	Varies from $\frac{1}{8}\ S$ to $\frac{1}{12}\ S$
28	Varies from $\frac{1}{10}$ to $\frac{1}{12}$
33	$\frac{1}{8}\ S$ to $\frac{1}{10}\ S$

TABLE 3. — DISTANCE BETWEEN CENTERS OF PLATE GIRDERS.

Specification Numbers.	Length of Span.	Distance c.c. Girders.
30		6½ ft. for deck, 13 to 14 ft. for through spans.
14, 15, 16	All spans	6½ ft.
11	All spans	6½ ft. and not less than depth
35	All spans	6½ ft. with open floor
21, 25, 26	0, 75 ft.	6½ ft.
16	Over 50 ft.	8 ft.
22, 28	60, 80 ft.	8 ft.
14	Over 70 ft.	8 ft.
15	Over 75 ft.	8 ft.
3	Over 80 ft.	8 ft.
35	All spans	8 ft. with solid floor
6	All spans	9 ft. with floorbeams and stringers
22	Over 100 ft.	9 ft.
21	Over 75 ft.	8 ft.
28	60 ft. and under	7½ ft.
28	80 ft. to 100 ft.	8½ ft.
3	under 80 ft.	6½ ft.
33	0, 70 ft.	6½ ft.
33	70, 100 ft.	spacing increases 6 in. for each 10 ft. in length
25, 26	Over 75 ft.	not less than ½ the span

TABLE 4. — CAMBER.

Specification Numbers.	Amount of Camber Required.
2	Equal to deflection under maximum load
16	¼ in. for every 24 ft. in length of span
36	¾ in. per 100 ft. in length of span
17, 30	one one-thousandth of span
22, 13	one fifteen-hundredth of span
25, 26–28	Sufficient to prevent sag

TABLE 5. — MATERIAL USED.

Specification Numbers.	Kind of Steel.
4, 11, 12, 16, 21, 30	Open Hearth; Soft, Medium and Rivet
7, 2, 8, 18, 19, 32, 15, 17, 32, 35, 24, 6	Open Hearth; Rivet and Structural
1, 5	Open Hearth; Rivet and Structural
13	Open Hearth; Medium
20, 22, 18	Open Hearth; Medium and Rivet
23, 31	Open Hearth; Soft
33	Open Hearth; Rivet, Railway Bridge and Medium
37	Open Hearth or Bessemer; Structural
14	Open Hearth or Bessemer; Soft, Medium and Rivet
36	Soft Steel
25, 26, 28	Open Hearth; Rivet, Medium and Structural

TABLE 6. — MAXIMUM PERCENTAGES OF ELEMENTS ALLOWED.

Specification Numbers.	Element.	Percentage Allowed.		
		Bessemer or Rivet Steel.	Acid Open Hearth.	Basic Open Hearth.
1	Phosphorous	0.0004	0.0004	0.0004
2, 17		. . .	0.0008	0.0005
4, 5, 11, 15, 18, 22, 30, 32, 33, 35, 28, 26, 25, 24, 13		. . .	0.0008	0.0004
5, Rivet Steel		0.0003	. . .	. . .
8; 21–23, 34		. . .	0.0006	0.0004
9		. . .	0.0006	0.0006
14		0.0001	0.0001	0.0001
16		. . .	0.0008	0.0006
18, 19, 32, 37, Rivet Steel		0.0004	. . .	. . .
19		. . .	0.0004	0.0008
20		. . .	0 0007	0.0005
31		. . .	0.0008	0.0008
37		0.0004	0.0004	0.0008
2	Sulphur	. . .	0.0006	0.0006
4, 5, 15, 18, 22, 35, 37, 24, 13, 25, 28, 26, 1		. . .	0.0005	0.0005
11		. . .	0.0005	0.0005
8, 18, Rivet Steel		0.0003	0.0006	0.0004
1, 32, 33, 37, Rivet Steel		0.0004	. . .	. . .

TABLE 7. — TEST REQUIREMENTS FOR ROLLED STEEL.

Rivet Steel.

Cold Bend.		Tension.							
Specification Numbers.	Ratio of Diam. of Curve to Thickness.	Specification Numbers.	Ultimate Strength, Lbs. per Square In.	Specification Numbers.	Elastic Limit, Lbs. per Square in.	Specification Numbers.	Per cent of Elongation in 8 Inch.	Specification Numbers.	Per cent of Reduction of Area at Point of Fracture.
1, 2, 11 . .				10		16, 21, 22			
12, 14, 13 . .		2, 14	48,000 to 50,000	5, 12	½ Ultimate	25, 26, 28	26	2, 15, 33	55
				14, 19, 21		12, 14			
18, 20, 26 .	0	4, 16	50,000 to 60,000	4, 33	60% Ultimate	2	50		
32, 33, 37 .				25, 26, 28		23, 33			
23, 10 . . .		5	45,000 to 55,000	30	55% Ultimate	4, 15, 35	28	20	54
25, 26, 28 .		34	48,000 to 54,000	11, 16	30,000	11, 34	30	34	60
15, rivets .	0	28, 26, 25, 23	48,000 to 56,000	15, 20 23	28,000	20	27	23	56
		4, 15, 21, 33							
. . . . .		12, 30	50,000 to 58,000	22, 13	26,000	7, 30, 1, 37, 32, 35, 5	1,500,000 Ultimate	30	2,800,000 Ultimate
. . . . .		10, 20	48,000 to 58,000	31	softest grade	10	1,400,000 Ultimate most ductile		
. . . . .		14	52,000 to 62,000			31			
. . . . .		13, 22, 35	46,000 to 54,000						
. . . . .		17	Not more than 55,000						
. . . . .		18, 19, 32, 37	45,000 to 55,000						
		1–7	46,000 to 54,000						
		24	Not more than 54,000						

TABLE 7. — TEST REQUIREMENTS FOR ROLLED STEEL (*continued*).

Medium Steel.

Specification Numbers.	Ratio of Diam. of Curve to Thickness	Specification Numbers.	Ultimate Strength, Lbs. per Square In.	Specification Numbers.	Elastic Limit, Lbs. per Square In.	Specification Numbers.	Per cent of Elongation in 8 Inch.	Specification Numbers.	Per cent of Red. c. on of Area at Point of Fracture.
11, 17	0	10 14, 16	60,000 to 70,000	10 12,14,21	50% Ultimate	13, 14, 16 25, 26, 28 17, 21, 4 5, 8, 12, 22	22	4, 5, 7, 14, 17	40
13 34, 20, 26		4	62,000 to 73,000	25, 26, 28 4, 8, 34, 30	55% Ultimate			11	43
30, 14, 36, 21, 33	1	5, 11, 17, 8	60,000 to 68,000	33	60% Ultimate	11	28	13	45
10, 25, 26, 28 18, 13, 35		21, 22, 12, 13		11	36,000	20, 13	22	20	44
12	3	25, 26, 28 30, 33	62,000 to 70,000	16	35,000	33, 34	25	22, 13	37
		9	57,000 to 65,000	13, 17, 22, 9	33,000			33	45
		34	56,000 to 64,000	20	35,000			34	50
		20	58,000 to 68,000			10	1,400,000 Ultimate		
		35	60,000			30	1,500,000 Ultimate	30	2,800,000 Ultimate

Drifting Test.

Specification Numbers.		
25, 26, 28, 5	Distance of center of hole from edge of plate Expressed in diameters of hole	2
4, 8, 17, 20, 30		1⅝
23		1½
34		1¼
8, 12	Enlargement of diameter of hole required	33⅓%
4, 5, 17, 20, 21 30, 15, 19, 31, 33, 23, 26 34, 25, 28		50%

TABLE 7. — TEST REQUIREMENTS FOR ROLLED STEEL (*concluded*).

Soft Steel.

Specification Numbers.	Ratio of Diam. of Curve to Thickness	Specification Numbers.	Ultimate Strength, Lbs. per Square In.	Specification Numbers.	Elastic Limits, Lbs. per Square In.	Specification Numbers.	Per cent of Elongation in 8 Inch.	Specification Numbers.	Per cent of Reduction of Area at Point of Fracture.
16, 9, 21 . .	0	1, 16	52,000 to 60,000	1, 12, 14, 21	50% Ultimate	1, 8, 12		4, 11, 14	50
23, 10 . . .						24	25	23	
18, 11, 14, 37		4	55,000 to 65,000	8, 30	55% Ultimate	14, 16, 21			
12	2	8	52,000 to 60,000	11	33,000	4, 11	26		
. . . . .		11, 12, 21, 30	54,000 to 62,000	16	32,000	37, 30	1,500,000 / Ultimate	30	2,800,000 / Ultimate
. . . . .		10, 23	52,000 to 62,000	23	28,000	10	1,400,000 / Ultimate		
				10	½ Ultimate				
Pin Steel .		23	62,000 to 70,000	23	33,000	23	17	23	40

Structural Steel.

2, 15, 18–1 .	0	2	60,000 to 70,000	28, 26, 25	55% Ultimate	25, 26, 28, 24	25	24	
				15	33,000	2	55	2, 15, 33	50
25, 26, 28, 33		24–1–35	56,000 to 64,000	19, 5	50% Ultimate	15	26	31	45
19, 31, 32, 7		15, 31, 33							
24		5, 18, 19,	55,000 to 65,000	31	58% Ultimate	31	27		
. . . . .		32, 37, 25,	58,000 to 66,000	33	60% Ultimate	33	28		
		26, 28, 7				5, 1, 35–7	1,500,000 / Ultimate		

TABLE 8. — DEAD LOAD OF FLOOR PER LINEAL FOOT OF SINGLE TRACK AND PER UNIT.

Materials.	Weight in Pounds.	Specification Numbers.
Rails and fastenings	100	3, 12, 16, 17, 27, 28, 35, 37, 25, 7, 26, 28
" " "	150	1
" " "	70	14
Rails, fastenings and guard rails	160	23
" " " " "	120	4
" " " " "	400	8, 11, 15, 31, 33, 30
" " " " "	450	36
Rails, fastenings and ties	500	19, 22, 13
" " " "	400	21, For girders 6½ ft. c.c.
" " " "	450	21, For girders 8 ft. c.c. 24
Rails, ties and timber	620	33, 32
Floor above stringers	600	34
Minimum weight of floor	400	12, 25, 26, 28
Timber deck, ballast track and fastenings	1,400	2, 35
Ballast per cubic ft.	110	12, 25, 26, 28, 35
Ballast " " "	120	15, 17, 32
" " " "	100	1
Timber per ft. b. m.	4.5	1, 2, 3, 12, 17, 28, 17, 23, 25, 28, 7, 26
" " " "	4	4, 35
" " " " creosoted	5	26
Oak per ft. b. m.	4.5	14, 25, 16, 26, 27
Yellow pine per ft. b. m.	3.5	14, 25
" " " " "	4	16, 26, 27
White pine and cedar per ft. b. m.	3	26
Wrought steel per cubic ft.	486	14
Concrete " " "	140	17
" " " "	150	35
Asphalt " " "	90	17

Train Loads.

Two coupled locomotives at head of train; Cooper's standard loading, E 40 minimum or two 50,000 lb. axle loads 7 W. c. c. Specification No. 1.

Standard loading; train of 3,200 lb. per lin. ft. headed by two 139-ton consolidation locomotives with 44,000 lb. on each of four axles 4½ feet apart. Heavy grade loading 50 per cent more. Specification No. 2.

5,000-lb. train load and two Cooper's E-50 locomotives; also consider 60,000 lb. on one axle. Specification No. 3.

4,000-lb. train load and two consolidation locomotives, each with four 40,000-lb. axles 4 ft. apart. Specification No. 4.

4,000-lb. train and two locomotives, each with two 45,000-lb. and two 50,000-lb. axles 5 ft. apart. Specification No. 7.

4,000-lb. train and two consolidation locomotives, each with four 42,000-lb. axles 5 ft. apart. Specification No. 8.

5,000-lb. train and two consolidation locomotives with four 60,000-lb. axles each 5 ft. apart. Specification Nos. 11, 33.

5,000-lb. train load and two Cooper's E-55 locomotives. Specification No. 13.

4,500-lb. train and two 142-ton consolidation locomotives (Cooper's E–40) or 4,500-lb. train and 60,000 lb. on each of two axles 7 ft. apart, and 5 ft. from first axle to train. Specification No. 15.

4,500-lb. train and two locomotives, each with 45,000 lb. on each of four axles 5 ft. apart, or Cooper's specifications, E-30, E-40, or E-50. Specification No. 16.

5,500-lb. train and two locomotives, each with 55,000 lb. on each of four axles 5 ft. apart, a 5,000-lb. train and spaces of 4 ft., 7 ft., 7 ft., and 9 ft. to axle loads of 30,000 lb., 68,000 lb., 68,000 lb., and 30,000 lb. Specification No. 19.

5,000-lb. train load and two locomotives, each with 45,000 lb. on each of four axles 5 ft. apart. Specification No. 34.

5,000-lb. train and two consolidation locomotives, each with 55,000 lb. on each of four axles 5 ft. apart. Specification Nos. 22, 21.

4,000-lb. train and two locomotives, each with 35,000 lb. on each of four axles $4\frac{1}{2}$ ft. apart. All plans 20 % excess. Specification No. 31.

5,000-lb. train and two locomotives, each with 50,000 lb. on each of four axles 5 ft. apart, or a 5,000-lb. train and one locomotive with 55,000 lb. on each of two axles 7 ft. apart. In both cases the stresses caused by these loads are to be multiplied by $(1\frac{1}{3} - L \div 300)$ where L is the loaded length and is less than 100 ft. Specification No. 32.

5,000-lb. train plus a 60,000-lb. concentrated load, the concentrated loads being increased 50 per cent for proportioning floor connections. Specification No. 30.

5,000-lb. train and two locomotives, each with 50,000 lb. on each of four axles 5 ft. apart. Specification Nos. 32, 35.

5,000-lb. train and two Cooper's E-50 locomotives. Specification No. 37.

5,000-lb. train and two Cooper's E-50 locomotives, or 130,000 lb. on two axles 7 ft. apart, or for ore lines 7,000-lb. train and two Cooper's E-50 locomotives. Specification No. 36.

4,000-lb. train and two 136-ton consolidation locomotives, together occupying 100 ft. of track, each having four 40,000-lb. axle loads 5 ft. apart; or if it will produce greater strains, a live load of 100,000 lb. on two axles 7 ft. apart. Specification No. 24.

5,000-lb. train and two 184-ton consolidation locomotives, each having four 52,000-lb. axle loads 5 ft. apart for track stringers, floorbeams and plate girders less than 110 ft. span. The maximum calculated stresses are taken from a table accompanying specifications.

5,000-lb. train load and two 177½-ton consolidated locomotives (Cooper's E-50) or 120,000 lb. on two axles, 6 ft. centers. Specifications Nos. 26, 28.

TABLE 9. — UNIT STRESSES.

Maximum Working Stresses in Pounds per Square Inch.

In Tension for Vertical Loads plus Impact.

Medium Steel.		Soft Steel.	
Specification Nos.	Stress.	Specification Nos.	Stress.
37	17,000	1	16,000
35	16,000		
In Tension for Vertical Loads, Impact, Wind and Centrifugal Force.			
	21,000		
3, 33, 25, 28, 26, 7, 23	16,000		
16, 21	17,000		
8, 19, 16, 21, 24	15,000		
11	12,000		
34, 11	10,000		
34	9,000		
35	20,000		

TABLE 10. — WEB SHEAR.

16, 21, 37–35, 7, 19	10,000	16, 21	9,000
3	9,000	23	12,000
17, 24, 13	6,000	25, 26, 28	$16{,}000 - 100\frac{d}{t}$
31	4,000		
33	11,000		
34	7,500		
15, dead	12,000		
15, live	6,000		

Live and Dead Load Stresses.

Stresses per square inch allowed for live load L, and for dead load D.

Specification 4, Flanges of rolled beams, 10,000 L, 10,000 D net. Flanges of plate girders up to 60 feet long, 10,000 L, 10,000 D net. Flanges plate girders over 60 feet long, 10,000 L, 20,000 D. Rivet shear, 10,000 L, 16,000 D. Girder webs, 7,000 L, 14,000 D.

Specification 15, Tension 8,000 L, 16,000 D.

Specification 32, Plates, shapes and bars, 11,000 L, 16,500 D.

Specification 11, Tension, 10,000 L, 20,000 D.

Maximum and Minimum Stresses.

Coefficients C used for values of unit working stresses determined by formula,

$$\text{Stress} = C\left(1 + \frac{\text{minimum}}{\text{maximum}}\right).$$

TABLE 11.

Specification Numbers ..	.... 30.....	14	17	$\left\{\begin{matrix}31\\22\end{matrix}\right.$	30	24
Coefficients (C).....	.....	9,500	8,500	8,000	7,000	7,500
	for compression $\left(7{,}000 - 26\frac{l}{r}\right)\left(1 - \frac{\text{min.}}{\text{max.}}\right)$			tension		

Impact.

Allowance for Impact to be Added to Live-Load Stresses.

$$I = S\left(\frac{300}{L + 300}\right)$$

where I = impact to be added to the live-load strain,
S = calculated maximum live-load strain,
L = length of loaded distance in feet which produces the maximum strain in member. Specification Nos. 1, 3, 21, 37.

Live-load stresses shall be increased in a proportion equal to the ratio between live-load stresses and the sum of dead and live load stresses. Specification No. 8.

$I = S\frac{10}{L + 10}$. Specification No. 11.

$I = L\left(\frac{L}{L + D}\right)$, where I is impact, L is maximum live-load stress, and D is dead-load stress. Specification Nos. 16, 35.

$I = L\left(\frac{L}{L + D}\right)$, where I is amount to be added for impact and vibration.

L is combined stress from live load and centrifugal force, D is dead-load stress. Specification No. 33.

Floorbeam and stringer connections 100 per cent live load, stringers and floorbeams 60 per cent live load. 15-ft. spans 60 per cent; 20-ft., 55; 30-ft., 45; 40-ft., 37; 50-ft., 30 per cent of live load; for spans of over 50 ft., 30 per cent of live load minus 1½ per cent for every additional ten feet. Specification No. 34.

Where there is no reversal of stress allowance, $I = \frac{L \times M}{M + m}$ when L is live load, M is maximum stress, and m is minimum stress. Allowance for momentum, tractive force, and centrifugal force = $\frac{1}{2} I$.

When maximum stresses are produced by loads on two or more tracks, I is reduced 20%. Specifications 25, 26, 28.

Centrifugal Stress.

Stress Due to Centrifugal Force of Train on Each Track.

$C = 0.03 \times W \times D$, for a curvature up to 5 degrees. Where C is centrifugal force in pounds, W is weight of train in pounds. D is degree of curvature. The constant 0.03 shall be reduced 0.001 for every degree of curvature above 5 degrees. Specification Nos. 2, 21.

A moving lateral force of 175 lb. for each degree of curvature up to 5 degrees, per lin. ft. of track. Specification No. 3.

For each train, $C = 0.02 \times W \times D$. Specification Nos. 4, 14.

Computed for a train speed of 60 miles per hour, increased for impact and vibration and assumed as applied 5 ft. above top of rail. Specification No. 8.

Centrifugal force in pounds, per lin. ft. of track, is 180 × degree of curvature. Specification No. 11.

Assumed to act 5 ft. above rail base and computed for 60 — 3 D miles per hour. Specification Nos. 12, 36.

$C = 0{,}045\ WD$ applied 5½ ft. above base of rail. Specification No. 13.

Centrifugal force assumed to be a live load applied 5 ft. above rail, $C = \frac{WV^2D}{85{,}700}$; v is velocity in miles per hour, assumed at 60 miles for curvature of 4 degrees or less; and $v = (60 - 2D)$ for curves of more than 4 degrees. Specification No. 15.

$C = WV^2 \div 32.2\ r$ where r is radius of curvature in ft. Specification No. 16.

Train velocity assumed 60 ft. per second, $C = 112\ W \div r$. Specification No. 17.

Assumed as moving live load equal to 0.02 of live load for each degree of curvature. Specification Nos. 19, 34.

Assumed to be applied $5\frac{1}{2}$ feet above rail base, C 0.045 $\times$ W $\times$ D. Specification No. 22.

A moving lateral force equals to 3% of live load on all tracks. Specification No. 30.

Computed for a velocity of 88 ft. per second for maximum train load. Specification No. 31.

Computed for a train velocity of 110 ft. per second. Specification No. 32.

$C = 0.04\ W \times D$. Specification No. 33.

$C = 0.05\ W \times D$. The coefficient 0.05 to be used for a curve of 1 degree, and must be reduced 0.005 per every degree of curvature above one, but shall never be less than 0.02. Specification No. 35.

$C = 0.02\ W \times D$ for curvature up to 5 degrees; for every degree more than 5 degrees, reduce coefficient 0.001. Specification No. 37.

Shall be treated as a live load. Specification No. 24.

$C = WV^2 \div 32.2\ r$ where r is radius of curvature in ft. Specifications 25, 26, 28.

TABLE 12. — MOMENTUM OF A SUDDENLY STOPPED TRAIN.

Assumed longitudinal horizontal force $\frac{V}{M}$ developed.

Specification Nos.	Value of M.
1, 2, 4, 7, 12, 14, 16, 13 . . .	0.2 WV, W is weight of train. V is velocity, and 0.2 is coefficient of sliding wheels.
3, 8, 23, 24, 37, 33, 1, 35, 19 .	20 per cent of live load.
11	1,000 lb. per lin. ft. of track.
12, 17, 21, 23	20 per cent of maximum tractive force of locomotives.
31, 32	25 per cent of live load when acting to increase stresses
25, 26, 28	15 per cent of live load when acting to decrease stresses

TABLE 13. — ASSUMED WEIGHT OF TRAIN PER LINEAR FOOT FOR DETERMINING THE ANCHORAGE FOR A LOADED STRUCTURE.

Specification Nos.	Pound per Ft.
13, 7, 25, 26, 28, 2, 12, 14, 16, 22, 38, 36, 23	800
3	2,000 in conjunction with all lateral forces.
11	1,000
19	600
15	800 per wind uplift

Wind Pressure in Pounds per Square Foot.

Assumed to act horizontally in either direction perpendicular to the vertical elevation of truss and floor and lateral systems.

30 lb. on bridge and on train 10 ft. in average height, and $2\frac{1}{2}$ ft. above rail base, moving across span, or, 50 lb. on bridge alone. Specification Nos. 4, 15, 17, 18, 19, 23, 37, 22, 7, (32, 3 ft. above base of rail.)

30 lb. for bridge and train plus 30 lb. on exposed surface of leeward truss assuming a minimum of 1,000 lb. per lin. ft. or 50 lb. per sq. ft. of unloaded structure. Specification No. 2.

600 lb. per lin. ft. loaded, or 200 lb. per lin. ft. unloaded flange. Specification No. 3.

30 lb. on twice side elevation plus 360 lb. per lin. ft. moving load from train surface, or, 50 lb. fixed load on twice side elevation of unloaded span. Specification No. 7.

30 lb. on bridge fixed, and 300 lb. per lin. ft. moving, for train surface. Specification Nos. 14, 8.

30 lb. fixed load plus 350 lb. moving train load applied 9 ft. above rail, initial stress of 10,000 lb. applied to all lateral and transverse braces. Specification No. 11.

150 lb. per lin. ft. fixed and 450 lb. moving load 6 ft. above rail base on unloaded span. Specification No. 12.

330 lb. per lin. ft. plus 30 lb. per lin. ft. for each foot in depth of girder live load, and 30 lb. on elevation plus 300 lb. per lin. ft. train surface, applied 8 ft. above rail base or 50 lb. on bridge surface. Specification No. 16.

300 lb. per lin. ft. dead load, and an additional 300 lb. per lin. ft. applied 8 ft. above rail for wind on train surface. Specification No. 21.

Moving load 400 lb. per lin. ft. ; dead load 200 lb. per lin.

ft. on loaded chord and 150 lb. on unloaded chord. Specifications 26, 25, 28.

400 lb. live load and 200 lb. per lin. ft. dead load at the loaded chord plus 200 lb. per lin. ft. at the unloaded chord making 800 lb. total. Specification No. 31.

For single-track spans, 300 lb. live and 150 lb. dead load per lin. ft. For double-track span, above loads increased 50 per cent, or, 30 lb. per sq. ft. of bridge elevation. Specification No. 33.

600 lb. per lin. ft. of loaded chord, or 300 lb. per lin. ft. of unloaded chord, live load. Specification No. 35.

30 lb. per square ft. of bridge for dead load plus 30 lb. per sq. ft. live load on train surface. Specification No. 34.

In top lateral system, 450 lb. live and 150 lb. dead per lin. ft., live load assumed to act on train surface 6 ft. above rail. Specification Nos. 36.

In bottom laterals, 200 lb. per lin. ft. Specification No. 36.

Moving load of 330 lb. per lin. ft. plus 30 pounds for each foot in depth of girder. On bottom laterals of deck spans, 150 lb. per lin. ft. Top laterals in deck bridges 450 lb. per lin. ft. Specification No. 24.

Shop Rivets.

TABLE 14. — STRESSES FOR SHOP-DRIVEN MACHINE RIVETS IN POUNDS PER SQUARE INCH.

Specification Nos.	Stress.	Specification Nos.	Stress.
Single Shear.		*Bearing.*	
1, 3, 23, 25, 28, 26–35	12,000	2, 25, 26, 28, 1, 35, 3	24,000
2, 14, 22, 13	7,500	31	12,000
8	12,500	2, 22, 36, 14, 13	15,000
19, 33, 21, 37, 1, 7	11,000	8	25,000
16	10,000	16	20,000
17, 36	9,000	17	16,000
31	6,000	19, 21, 33, 37, 7	22,000
32, live load	7,000	14	13,500
32, dead load	10,500	23	26,000
		32, live load	13,000
		32, dead load	20,000
Bending.			
2	18,000		
25, 26, 28, 7	24,000		
19	22,000		
34	15,000		

Field Rivets.

TABLE 15. — EXCESS OF FIELD RIVETS OVER SHOP RIVETS.

Specification Nos.	Per cent of Excess.
13, 11, 4, 18, 14, 37, 23, 24, 35, 19	25 per cent Hand driven
1, 3, 15, 16, 17, 22, 30, 32, 33, 25	20 " " " "
1, 23	10 " " Machine driven
36	50 " " Hand driven
25, 26, 28	25 " "

Web Rivets.

Shearing and Bearing Stresses for Proportioning Web Rivets.

Shear on abutment side is assumed to be transferred to flange angles in distance equal to depth of girder. Specification Nos. 1, 2, 4, 7, 21, 31.

Flange web rivets proportioned to transmit direct flange load, plus total shear in distance equal to depth of girder at that point. Specification Nos. 3, 11, 18, 33, 35.

Web flange rivets transmit force equal to shear at any point in a distance equal to depth of girder measured from point towards center of span. Specification Nos. 6, 30, 34.

CHAPTER IV

FLANGES, WEBS, STIFFENERS, BRACING, EXPANSION, BEARINGS.

Flanges. At least one half the cross-sectional area of the flange shall be made with angles, or the largest size angles shall be used. Flange cover plates shall be of uniform thickness, or shall decrease in thickness from the angles outwards. Specification Nos. 21, 22, 33, 35, 2, 3, 4, 8, 11, 12.

The Gross Area of both flanges is required to be the same in specifications Nos. 1, 12, 14, 16, 17, 19, 21, 30, 33, 24, 25, 26, 28, 35, 11.

Flange Sections.

All top flanges shall have one full length cover plate; deck girder top flanges shall have more than one full length cover plate if necessary to avoid housing the ties more than 1 in. The cover plates shall not extend more than 5 in. beyond the outer rows of flange rivets except in long through girder top flanges where they may extend $6\frac{1}{2}$ in. beyond the flange rivets if the plates are connected by a row of rivets each side beyond the flange angles. Compression flanges shall be stayed against transverse crippling when their length is more than sixteen times their width. Specification No. 2.

The width of flanges shall not be less than $\frac{1}{12}$ the distance between side supports. Flange plates must extend at least two rivets beyond their theoretical length at each end. The first plate must be full length. Specification No. 3.

Cover plates should not project more than eight times their thickness beyond the outer rivet row. Where there are four rows of flange cover plate rivets their pitch should not be more than sixteen times the thickness of the thinnest plate, nor less than three times the diameter of the rivet. Specification No. 4.

Flange cover plates more than 12 in. wide shall have four lines of rivets. Specification No. 6.

Where flange plates are used, one cover plate of each flange shall extend the whole length of the girder.

Through plate girders shall have their top flanges stayed at

each end of every beam, or, in case of solid floors, at distances not exceeding 12 ft. by knee braces or gusset plates. Specification No. 7.

Flange plates must not extend more than 4 in. or more than eight times the thickness of the outside plate beyond the rivet lines. The unsupported length of compression flanges must not exceed fifteen times their width. Splices in cover plates and angles must be located where the sectional area is in excess of the theoretical requirements, and must be 25 per cent stronger than the section spliced. Specification No. 8.

Preferably without plates ; no plate shall be thicker than the angle. First plate must be full length, and other plates must extend 12 in. beyond their theoretical ends. Projections of plates beyond outside rows of flange angle rivets must not be more than 4 in. nor more than eight times the thickness of thinnest plate. No two pieces of the flange shall be spliced in the same vertical plane, and the cross-sectional area at splices must be greater than is theoretically required. The splice must be 20 per cent stronger than the piece spliced, and two additional rivets on each side of the joint must be allowed for each plate intervening between splice and splice plate. Specification No. 11.

Flange plates must extend beyond their theoretical length two rows of rivets at each end. Compression flanges must be stayed against transverse crippling when their length is more than sixteen times their width. The distance between rivets in compression flange plates shall not exceed thirty times their thickness. Plates must not extend beyond the outer rows of flange angle rivets more than 5 in. or eight times the thickness of the first plate. Specification No. 12.

Angles shall form as large a part of the flange area as practicable, preferably not less than one half. Only one flange piece shall be spliced at the same point ; splices shall break joints and shall be made with angles in pairs, both sides of the flanges. Flange plates shall be of uniform thickness or shall diminish in thickness outwards from the flange angles. The first flange plates on top and bottom flange shall extend the full length of the girder except for girders less than 75 feet long, where the bottom flange plate need not extend the full length. Specification No. 15.

Compression flanges shall be stayed transversely when their length is more than twenty times their width. Angle legs 6 in. or more in width shall be connected to the web plate by two

rows of staggered rivets with a pitch of not less than $2\frac{1}{4}$ in. nor more than $4\frac{1}{2}$ in. Specification No. 16.

Compression flanges shall be stiffened if their unsupported length exceeds twenty times their width. Flanges carrying heavy concentrated moving loads, where cover plates are not used, must have throughout the uniform minimum rivet pitch required at the ends. Where two or more cover plates are used they must be of equal thickness or decrease in thickness from the angles outwards. The cover plate shall not extend more than 5 in. or more than eight times the thickness of the plate beyond the outer row of rivets. Cover plates more than 14 in. wide shall have four lines of rivets. Rivet pitch shall not be more than 16 times the thickness of thinnest plate, nor less than three times diameter of rivet. Specification No. 17.

Top flanges shall generally have one full length cover plate. Where top flange plate is not full length the web plate shall be flush with the backs of top flange angles. Flange plates shall extend to include four pairs of rivets beyond points where the increase of section is required. Plates shall not extend more than eight times the thickness of first plate beyond the outside flange angle rivets. When two or more plates extend 3 in. or more beyond the flange angles they shall have a row of rivets of not more than 9 in. pitch, outside the angles. No plate shall be thinner than any exterior to it. Specification No. 18.

Cover plates must be at least 12 in. longer than their net calculated length. When over 14 in. wide they shall have at least four lines of rivets with a maximum pitch of 9 in. each. The unsupported length of the flange must not exceed twelve times its width. Specification No. 19.

Compression flanges must be stayed transversely at distances not exceeding twelve times their width. Angles 6 in. or more in width must have two rows of staggered rivets through the web plate. Cover plates more than 13 in. wide must have in each flange angle two rows of staggered rivets with a maximum pitch of $4\frac{1}{2}$ in. Rivet pitch at the ends of the cover plates shall not exceed four diameters of the rivet for a distance equal to twice the width of the plate. Through girders must have end cover plates, rounded shore ends of top flanges and at least one full length top flange cover plate. All except the first cover plates must be long enough to have at least two rows of rivets beyond the theoretical length. Specification No. 21.

Plates must not extend more than 5 in. nor more than nine

times the thickness of the first plate beyond the outer row of flange angle rivets. Cover plates more than 14 in. wide must have four lines of rivets. When practicable the first top flange cover plate must be full length. Rivet pitch must not be more than 6 in. nor more than sixteen times the thickness of the thinnest plate nor less than three times the diameter of the rivet. Specification No. 22.

Flange areas required must be contained as far as practicable in the angles. First cover plate must extend full length. Full section cover plates or angles must be used at splices. No two splices must occur directly opposite in the same flange section. Cover plates will generally be 14 in. wide with 6×6 in. angles, the latter with two rows of staggered rivets through each leg. When two or more flange plates project more than 3 in. beyond the flange angles the projections shall have extra lines of rivets of 9 in. maximum pitch. At the shore ends of through girders the top flange angles and plates will be continued to the lower flange and will be curved to 18 in. radius for spans up to 50 ft., 21 in. up to 60-ft. span, and 24 in. for longer spans. Specification No. 30.

In girders of less than 50-ft. span the vertical legs of the flange angles shall not be considered effective in computing flange sections. At least one flange plate must be full length; the other cover plates must have, beyond the theoretical required length, enough rivets to develop the allowed strength. The maximum pitch for staggered rivets in the outside row of a cover plate is 9 in. Specification No. 31.

In girders with flange plates at least half the area of each flange shall be angles, or $6 \times 6 \times \frac{5}{8}$ in. angles shall be used. Compression flanges shall be stayed transversely when their length exceeds fifteen times their width. Specification No. 32.

All through plate girders shall have end cover plates, and where their top flanges project 3 ft. or more above the ties they shall be rounded at the ends to circular curves with the centers at about the level of the top of the ties. Angles shall have large cross-sections, and shall generally have two rows of staggered rivets in each leg. Cover plates shall not be thicker than the angles, and preferably not more than $\frac{3}{4}$ in. thick. The first cover plate must always be full length, and others must extend 12 in. beyond theoretical length at each end. Cover plates must not extend beyond the outer rows of angle rivets

more than 4 in. nor eight times the thickness of the outside plate. Specification No. 33.

Width of compression flange must not be less than $\frac{1}{12}$ the distance between its side supports. When the grip of the rivet exceeds its diameter the number of rivets must be increased one per cent for each additional $\frac{1}{16}$ in. of grip. Flange angles must have a minimum thickness of $\frac{1}{10}$ the width of the outstanding leg. The first top flange cover plate must be full length. The top flanges of through girders must be stayed by kneebraces or gusset plates at every floorbeam, or, for solid floors, at maximum distances of 12 ft. Specification No. 35.

Cover plates must be 3 ft. longer than theoretical length. Specification No. 36.

Preferably at least one plate in each flange; first plate in top flange to be full length. Both flanges same gross area. Longitudinal spacing of staggered rivets not less than 2 in. Flange plates of equal thickness or decreasing outwards, and long enough for at least two rows of transverse rows of rivets beyond theoretical ends of plates. Flange angle splices to be made with cover angles. In flange splices the joints of different parts to be broken. Specification No. 24.

Flange plates more than 14 in. wide, or projecting more than 3 in. beyond the edges of the flange angles, must have at least four lines of rivets. There must be at least two rows of rivets at the regular pitch beyond the theoretical ends of cover plates. The first cover plate must be full length. Specification No. 23.

Web Stiffeners.

When the thickness is less than $\frac{1}{80}$ the unsupported depth web stiffeners shall be spaced $1\frac{1}{2}$ times the depth. Specification No. 2.

The two pairs of end stiffener angles and fillers shall be proportioned to take the maximum end shear. Intermediate stiffeners, in pairs, shall be spaced at all concentrated loads, and at a distance about equal to the depth of the girder or a maximum of 6 ft. The size of the stiffener angles shall increase with the depth of the girder, $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ in. for a $6\frac{1}{2}$-ft. girder, $5 \times 3\frac{1}{2}$ in. for 7 to $7\frac{1}{2}$ ft. and $6 \times 4 \times \frac{3}{8}$ in. for 8 feet and over. Specification No. 3.

All shop-riveted stiffener angles must bear tightly against top and bottom flange angles. Filler plates with ends bearing against flange angles must be used under end stiffener angles.

Intermediate stiffener angles shall preferably be offset, except that fillers may be used under angles connecting floorbeams to girders, in which case fillers must bear on bottom flange angles but may have $\frac{1}{8}$-in. clearance at top. Specification No. 36.

Stiffeners at bearings and for top flange concentrated loads may be fitted to top flange and assumed to take 8,000 lb. load per square in. of contact surface where it is not convenient to secure sufficient rivet-bearing strength, otherwise there shall be $\frac{1}{8}$-in. to $\frac{1}{4}$-in. clearance between the flanges and the ends of the stiffeners. The rivet pitch in stiffeners shall not be greater than in the adjacent portions of bottom flanges. Specification No. 6.

All stiffeners have fillers and are milled at the ends to fit flange angles. They are placed over bearings and at concentrated loads, and are spaced not farther apart than the depth of the girder near the ends, nor more than twice the depth of the girder apart near the center. Specification No. 4.

Webb stiffeners shall be in pairs. Those over the end bearings shall be on fillers, the outstanding legs shall be as wide as the flange angles will allow and they shall be brought to a close bearing against the upper and lower flange angles. Intermediate stiffeners may be on fillers or offset over the flange angles. Their outstanding legs shall not be less than $\frac{1}{30}$ of the depth of the girder plus 2 inches. The thickness of all stiffeners shall not be less than $\frac{3}{8}$ inch, and the rivet pitch in them shall not be over 5 inches.

Stiffeners shall be riveted to the web at all points of concentrated loads, and also, when the thickness of the web is less than $\frac{1}{60}$ of the unsupported distance between flange angles, at points throughout the length of the girder, generally not further apart than the depth of the full web plate, with a maximum limit of 6 feet.

The stiffeners at bearings and other points, where they carry load direct, shall be connected to the web with a sufficient number of rivets to transmit the strain from the concentrated load, and shall be proportioned for this strain by the formula $S = 16{,}000 - 70 \frac{\text{length}}{\text{radius of gyration}}$. The effective length shall be assumed to be one-half the length of the stiffener. Specification No. 7.

Stiffeners at end bearings and at all points of concentrated

loading shall be proportioned as columns to carry the total shear and provide amply for its distribution into the web. Specification No. 8.

At the end bearings two pairs of stiffener angles with fillers and included portion of web shall have a column section sufficient for the maximum shear. Where the unsupported depth of web exceeds 50 times its thickness intermediate stiffener angles in pairs will be put at points of concentrated loading, at web splices, and at cross frames, and will have a maximum spacing of $5\frac{1}{2}$ ft. or equal to the depth of the girder in the middle or one-half that distance at the ends. Intermediate stiffeners shall be crimped except at floorbeams, cross frames, etc. Sizes vary with depth of girder, from $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{16}$ in. for 4 ft. and under to $4 \times 3 \times \frac{3}{8}$ in. from 4 to 7 ft. and $5 \times 3 \times \frac{3}{8}$ in. for girders more than 7 ft. deep. Specification No. 11.

Stiffeners at end bearings to have fitted ends. Intermediate stiffeners to be used at points of concentrated loading and spaced not farther than the depth of the girder nor a maximum of 5 ft. where the shear per square in. exceeds 10,000-75 times ratio of depth of web to its thickness. All stiffeners are proportioned to carry maximum shear with a unit stress of 10,000-45 $(1 \div r)$, where 1 is the length of stiffener, and r is radius of gyration. The web rivets in each stiffener are proportioned for the maximum shear. Specification No. 12.

Stiffeners must be used at intervals equal to the depth of girder or track stringer whenever the ratio of the unsupported depth of the web to its thickness exceeds fifty; also at points of concentrated loads. There shall be two pairs of stiffeners at the end bearings of girders, these four stiffeners to take care of the maximum end shear. Specification No. 13.

Stiffeners with fitted ends at end bearings, points of concentrated loading and spaced about the depth of the girder wherever the maximum shear per square in. exceeds $12{,}000 \div [1 + (d^2 \div 3{,}000\ t^2)]$, where d is the depth between flange angles, and t is thickness of web. Intermediate stiffeners crimped for girders more than 30 in. deep. The sizes of the stiffener angles increase with the depths of the girders as follows: For girders up to 5 ft. deep, with 5-in. flange angles, $4 \times 3 \times 5\frac{5}{16}$-in. end stiffeners, and for 6-in. flange angles, $5 \times 3 \times \frac{3}{8}$-in. end stiffeners; intermediate stiffeners, $3\frac{1}{2} \times 3 \times \frac{5}{16}$-in.; for 5-ft. to 7-ft. girders, $5 \times 3\frac{1}{2} \times \frac{3}{8}$-in. end stiffeners, and $4 \times 3 \times \frac{5}{16}$-in. intermediate stiffeners; for girders more than 7 ft. deep, $5 \times 3\frac{1}{2} \times \frac{7}{16}$-in.

end stiffeners, and 5 × 3 × $\frac{5}{16}$-in. intermediate stiffeners. Specification No. 14.

Stiffeners with fillers at the end bearings, at all points of application of concentrated loads and intermediate stiffeners with a spacing of

$$60\,t\sqrt{\left(\frac{12{,}000}{s}-1\right)}$$

where s is the shear in pounds per square in. of gross area of web plate, and t is the thickness of the web. The intermediate stiffeners may be omitted if this spacing is greater than the clear distance between girder flange angles. The end stiffeners have their cross-section proportioned for the maximum bearing stress. The maximum distances between stiffeners must not exceed 6 ft. Specification No. 15.

Pairs of stiffeners and fillers at both edges of end bearings, at concentrated loadings and intermediately at maximum distances of 6 ft. spaced so the shear per ft. shall not exceed $15{,}000 \times 12\ t \div [1 + (d^2 \div 3{,}000\ t^2)]$, where t is the thickness of the web, and d is the distance between its supports. Intermediate stiffeners shall be used where the ratio of unsupported depth of web to thickness is greater than 50. Specification No. 16.

Stiffeners with milled ends at bearings and points of concentrated loading and intermediate stiffeners spaced the depth of the girder where the shear per square in. exceeds $12{,}000\ [1 + (H^2 \div 3{,}000)]$, where H is the ratio of depth of web to thickness. Specification No. 17.

Stiffeners shall be in pairs with their ends fitted to the girder flange angles. End stiffeners shall be proportioned so that the stress from the maximum shear shall not be greater per square in. on their cross-sectional area, including that of the portion of the web between them, than is allowed in the top flange of the girder. Stiffeners will be placed at concentrated loads and intermediately at a maximum distance of 5 ft. or of the full depth of the web plate where the thickness of the latter is less than $\frac{1}{60}$ of its unsupported length between flange angles. Specification No. 18.

Pairs of web stiffener angles with fillers shall be spaced the depth of the girder where the thickness of the web is less than $\frac{1}{30}$ the distance between flange angles. Over each bed plate there shall be two pairs of stiffeners with a combined cross-sectional area equal to the total end shear, including impact, ÷

15,000. All stiffeners shall have the ends fitted to the flange angles. Sizes of stiffener angles shall vary from $3 \times 3\frac{1}{2} \times \frac{3}{8}$ in. for 30-ft. girders to $5 \times 3\frac{1}{2} \times \frac{3}{8}$ in. for 90-ft. girders. Specification No. 19.

Stiffeners with fillers fitted tightly to flange angles at concentrated loads and intermediately spaced less than the depth of the girder or a maximum of 5 ft. where the thickness of the web is less than $\frac{1}{60}$ the distance between girder flange rivets. The sizes of the intermediate stiffener angles are governed by the ratio of the depth of the web to its thickness; for ratios of 60 to 150 the stiffeners are $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ in.; for 150 to 220, $4 \times 3\frac{1}{2} \times \frac{3}{8}$ in.; for 220 and over, $5 \times 3\frac{1}{2} \times \frac{3}{8}$ in. There shall be at least two pairs of stiffeners at the end bearings, which, with their fillers, are proportioned for the maximum end shear. Their rivet pitch must not be more than $1\frac{1}{2}$ times the flange rivet pitch at the ends, nor more than 5 in. Specification No. 21.

At each end bearing two pairs of stiffeners proportioned for maximum shear; stiffeners at concentrated loads and intermediately spaced the depth of the girder wherever the ratio of unsupported depth of web to its thickness exceeds 50. Specification No. 22.

All stiffeners to support the full width of flange angles and to have fillers. Stiffeners to be placed at both ends of bearing plates and at all web splices, and intermediately at distances equal to height of girders, except at ends where they shall be spaced $\frac{1}{2}$ height of girder when the unsupported distance between flange angles exceeds 50 times the thickness of the web plate. Specification No. 30.

Where the thickness of the web plate is less than $\frac{1}{60}$ of the unsupported distance between flange angles, stiffeners shall be riveted on both sides of the web 3 ft. apart for girders less than 3 ft. deep, and for deeper girders, generally not exceeding the depth of web plate up to a maximum of 6 ft. Stiffener angles must bear top and bottom, and be milled to fit. Specification No. 31.

Stiffening angles shall be placed at the ends of the girders and at concentrated loads, designed to carry the entire loads at these points. Intermediate stiffeners shall be placed not more than 4 ft. apart at the ends or 8 ft. at the center. Stiffener angles shall be closely fitted to the bottom flanges of the girders at all bearing points and to the top flanges throughout. Filler plates

shall be fitted to the flange angles at all bearing points. Specification No. 32.

Stiffeners fitted to the flange angle fillets are placed at the end bearings and at concentrated loads. Intermediate stiffeners spaced a maximum of 5 ft. or the depth of the girder in the middle and $\frac{1}{2}$ the depth at the ends are required where the unsupported distance between flange angles is more than 50 times the thickness of the web plate. The sizes of intermediate stiffener angles vary with the lengths of the angles from $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ in. for girders up to 50 ft. to $4 \times 3\frac{1}{2} \times \frac{3}{8}$ in. for 50 to 70-ft. girders, $5 \times 3\frac{1}{2} \times \frac{3}{8}$ in. for 70 to 90-ft. girders, and $6 \times 3\frac{1}{2} \times \frac{3}{8}$-in. angles for girders more than 90 feet long. Specification No. 33.

Web stiffener angles shall be in pairs, and those at end bearings shall have fillers and shall be fitted to bearing against the flanges. Intermediate stiffeners may have fillers or be crimped; their outstanding legs shall be not less than $\frac{1}{3}\frac{1}{0}$ the depth of the girder plus 2 in. All stiffeners shall have a thickness of not less than $\frac{3}{8}$ in. and a rivet pitch of not more than 5 in. Intermediate stiffeners shall be placed at concentrated loads and at such points as are required by the formula, shearing stress per square in. of gross section of web $= 12{,}000 - 40\,(d \div t)$, where t is thickness of web, and d is distance between stiffeners. Where stiffeners carry load directly, their web connection rivets are proportioned to transmit it all, and their cross-sectional areas are determined by the formula, Unit stress $= 16{,}000 - 70\,(l \div r)$ where l is length, and r is least radius of gyration. The effective length is assumed to be $\frac{1}{2}$ the actual height of the stiffener. These dimensions shall not be less than the minimum dimensions previously given. Specification No. 35.

Vertical web stiffener angles shall be riveted to both sides of the girder whenever the unsupported distance between the flange angles exceeds fifty times the thickness of the web plate. Stiffeners shall be symmetrical about the center of the girder and not farther apart than the distance between centers of flange angles. If unequally spaced, the spacing shall decrease from center of girder to ends. There shall be a pair of stiffeners at each end of each bed plate, at all web splices, and at intermediate points as required. The ends of all stiffeners, fillers and web splice plates shall fit tight. Specification No. 24.

Web stiffeners with close bearings on top and bottom flange angles shall be located at both ends of the bearing plates, at all

points of local and concentrated loads, and shall be spaced not farther apart than the depth of the girder nor more than a maximum of 5 ft. wherever the thickness of the web is less than $\frac{1}{60}$ of the unsupported distance between flange angles. Specification No. 23.

Bracing.

Through plate girders shall have kneebraces or gusset plates at the ends and at each floorbeam. Specification Nos. 1, 19, 25.

Deck girders shall have sway frames composed of at least four angles at the ends and at intermediate points 12 to 14 ft. apart. Top lateral angles shall not be smaller than $6 \times 4 \times \frac{5}{8}$ in. with six rivets at each connection. Through girders shall have kneebraces from top flanges to floorbeams at intervals not exceeding twelve times flange width. Lower lateral systems shall have substantial end struts as deep as the stringers will permit, and diagonal angles riveted together at their intersection and to stringers in each panel and connected to gusset plates engaging main girders and lower flanges of floorbeams. Specification No. 3.

Through plate girder top flanges shall be stayed laterally at distances not exceeding twelve times their width or 16 ft. For spans of 35 ft. or more they shall have top and bottom flange lateral systems. They shall have stiff cross-frames at the ends of the spans and intermediately at intervals of not more than 20 ft. These frames shall be proportioned to resist unequal loading of the girders, and those at the ends shall be of the same strength as the top lateral bracing. Through plate girders shall have lateral bracing between floorbeams and gusset plates at floorbeams, or at intervals of not more than 12 ft. if there are no floorbeams. All laterals shall be angles. Specification No. 11.

Deck plate girder spans shall have cross-frames or diaphragms at the ends and not more than 16 ft. apart intermediately. Specification No. 13.

Through plate girders will have gusset plates forming parts of the floorbeam webs and riveted between their flange angles and stiffened with a pair of $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$-in. flange angles. In deck girders of less than 35-ft. span the bottom laterals may be omitted. All deck spans have cross-frames at the ends and at intermediate intervals of not more than 20 ft. Lateral and sway brace angles must not be less than $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ in. and

must have not less than four rivets in each end. Specification No. 15.

All deck spans shall have end sway braces proportioned to transmit accumulated upper lateral stresses to substructure. There shall be intermediate sway bracing at alternate panel points of main laterals except for deep girders where they are at every panel point. Where floorbeams are framed between deck girders kneebraces may be used below them in place of sway braces. Except on curves lower lateral braces may generally be omitted in spans up to 50 ft. Specification No. 18.

Through spans will have braced gusset plates with two angles at each edge at all floorbeams and struts. Deck spans will have sway frames of at least four angles at the ends and at intermediate points 12 or 15 ft. apart. Specification No. 21.

Deck spans shall have cross-frames at each end and at intermediate spaces of not more than 16 ft. Specification No. 22.

Lateral and centrifugal stresses tending to distort cross-sections must be provided for by sway bracing in deck bridges and by gussets in through spans. Lateral braces must have flange connections of greater strength than themselves. Through spans shall have a full system of adjustable horizontal lateral bracing in each panel, using floorbeams for struts when practicable. Deck spans must have complete top and bottom horizontal lateral bracing and diagonal sway bracing, the latter proportioned at the ends to transfer the lateral stress to the plane of the end posts. Connections must be designed to place pins in double shear. In through spans the gusset plates must form part of the floorbeam webs and be riveted to the girder web-stiffeners to brace the flanges. The width of the narrow end of the gusset will equal that of the girder inside flange angle. When the length of the inclined edge of the gusset is more than ninety-six times its width a 6-lb. angle $\frac{3}{8}$ in. thick must be used as a stiffener. When the floor beams are near the middle height of girder the gussets will run to each flange. Specification No. 30.

Deck spans must have sway bracing at each end and at intermediate intervals not exceeding 40 ft. where there are both top and bottom laterals, and at intervals not exceeding 20 ft. where the bottom laterals are omitted. Specification No. 32.

The lengths of lateral bracing panels must not exceed twelve times the width of the flange. Specification Nos. 33, 25, 26, 28.

Deck spans must have transverse bracing at each end

proportioned to carry lateral stresses to supports. The lateral angles shall have a minimum size of $3\frac{1}{2} \times 3 \times \frac{3}{8}$ in. with four rivets in each end connection. Specification No. 35.

Gusset plate braces, to support the top flanges of half-through girders, shall be placed at all floorbeams; where end floorbeams are not used, the gussets shall connect to substantial end struts. The outstanding edges of the gusset plates shall be stiffened with angles on both sides of the plates. Where the base of the gusset brace is narrow, the plate shall extend between the top and bottom flange angles of the floorbeam or strut, to comprise a part of the web of the same, and the angles at the outer edge of the gusset shall be substantially connected to the top flange of the cross-floorbeam or strut. Specification No. 18.

Web Moment.

TABLE 16. — PROPORTION OF WEB AREA ASSUMED TO RESIST BENDING MOMENT, I.E., TO ACT AS PART OF CROSS-SECTION OF EACH FLANGE.

Specification Nos.	Proportion of Web Area.	Remarks.
21, 37, 35	$\frac{1}{8}$	
2	$\frac{1}{8}$	when there are no web stiffeners.
30, 4, 7, 11, 12, 33, 24, 15	0	
31	0	for spans up to 90 ft.
14, 16, 17	$\frac{1}{8}$	
6	$\frac{1}{8}$	if web is in single piece.
31	$\frac{1}{8}$	for spans of over 90 ft.
25, 26, 28		proportioned by least moment of inertia of net section

TABLE 17. — KINDS OF BEARINGS FOR VARIOUS SPAN LENGTHS.

Specification Nos.	Spans.	Specification Nos.	Spans.
Roller Bearings.		*Sliding Ends.*	
1, 12, 11, 8, 37, 23, 24, 17	Over 80 ft.	1, 8, 12, 35, 38, 24, 23 17,	0 ft. to 80 ft.
2	" 106 ft.	2, 36	0 ft. to 70 ft.
3, 35, 28, 26, 25 (Rollers or Rockers)	" 80 ft.	14, 19, 21, 22, 25, 26, 28 33, 15–13	0 ft. to 75 ft.
25, 26, 28, 15, 33 . . .		30	0 ft. to 60 ft.
4, 14, 18, 19, 21, 22, 23, 34	" 75 ft.	*Pin Bearings.*	
30, 31, 32	" 60 ft.	12, 11, 18, 35, 37, 24, 28, 26	Over 80 ft.
36	" 70 ft.	4, 15, 21, 22, 23, 33, 13 .	" 75 ft.
		31	" 60 ft.
		36	" 70 ft.

Expansion.

An Expansion of 1 in. in 100 ft. is provided for by specifications Nos. 3, 4, 11, 33, 35, 15. An expansion of $1\frac{1}{4}$ in. in 100 ft. is provided for in specifications Nos. 25, 26, 28 and 1 in. in 70 ft. for specification No. 30.

A Temperature Variation of 150 degrees Fahr. is provided for by specifications Nos. 13, 4, 12, 14, 18, 19, 22, 37, 23, and of 120 degrees by specification No. 4, 180 degrees by specification No. 24.

Lead.

Lead Plate Bearings are required by specifications Nos. 15, 32, 36, 24, 21, 7.

Corners.

Rounded Upper Corners are required at the shore ends of through plate girder bridges by specifications Nos. 3, 4, 15, 18, 26, 28, 25, 33, 31.

Roller Sizes.

TABLE 18. — FORMULÆ FOR PROPORTIONING EXPANSION ROLLERS.
P = pressure per lin. in. of roller, d = diameter of roller.

Specification Nos.	Formula.	Minimum diameter.
2	$P = 300\,d$	4 in. for cylindrical 6 in. for segmental
3, 11, 18, 35	$P = 600\,d$	4 in.
12	$P = 300\,d$	$2\frac{7}{8}$ in.
14, 34	$P = 700\sqrt{d}$ for soft steel	. . . .
14, 34	$P = 900\sqrt{d}$ for medium steel	. . . .
8	. . . . .	$3\frac{3}{8}$ in.
15, 31, 33	. . . . .	3 in.
17	$P = 500\,d$	$3\frac{1}{2}$ in.
21	. . . . .	$3\frac{1}{2}$ in.
19	$P = 1{,}200\sqrt{d}$	. . . .
30, 11, 13	$P = 400\,d$	. . . .
35, 36, 25, 26, 28, 33	$P = 600\,d$	. . . .
14	. . . . .	$2\frac{1}{2}$ in.
24, 31	$P = 300\,d$	. . . .
23	$1{,}000\sqrt{d} + 100\,d$	3 in.
8, 26, 25	. . . . .	3 in.
34	$900\sqrt{d}$	

Bed Plates.

TABLE 19. — PRESSURE ON BED PLATES IN POUNDS PER SQUARE INCH

Specification Nos.	Pounds.
37, 17	300
12, 22, 31, 36, 30	250
32	250 live, 400 dead
3	500 on granite
22, 24, 13	300 on masonry
23	500
33, 35, 25, 26, 28, 19	400

CHAPTER V

EXTRACTS FROM SPECIFICATIONS

Besides the points covered by the foregoing summary of requirements, some of the provisions that have been mentioned are modified or amplified, and some additional ones and special features are included in various specifications. In some cases they are due to peculiarities of the traffic, to characteristics required for the structures, to the structural or mechanical standards involved, to efforts to develop the general construction, or even to personal choice and opinion. In general they include the labor-saving tabulation of frequently used data, and the less vital of the requirements for calculations, design, and shop work, which are gradually adopted to raise the standards, and in some instances are essentially used by individual engineers in their personal execution of the work without being formally adopted. The extracts here given, together with the preceding summaries and comparisons, make an approximately complete outline of the essentials of current railroad specifications.

These extracts are made in the sequence of reference enumerations of the specifications, and the credit of any clause to a certain specification does not imply that it occurs in that specification only, but that it occurs there first in the list, or is more satisfactorily expressed there than elsewhere.

Although the specifications from which the preceding abstracts have been prepared were the latest available at the time of compilation, some of them have been written several years and do not completely cover the present practice, which in some instances is being modified so tentatively or insensibly that the engineers do not like either to announce absolute departure from or complete adherence to the precise text. It is, however, conceded by most structural engineers that, at the date of this writing, 1907, rapid and important developments are being made in specifications tending principally towards uniformity of requirements and a single quality of steel for ordinary bridge work, so that the practice of to-day does not fully agree with that recently accepted, or with what may be reasonably expected in a very few years.

Recognizing this fact the American Railway Engineering and Maintenance of Way Association appointed in 1903 a committee of eminent bridge and railroad specialists to prepare a standard specification. Materials, workmanship, unit strains, and proportion of parts were successively considered; and among the most important features of the new specifications were the use of a single quality of soft open hearth steel, suitable for both tension and compression members and for punching, up to thick plates without reaming, and the adoption of Cooper's uniform loading. On several points which it was believed not practicable or probable that uniform practice could be accepted, desirable requirements were suggested and special blanks were left in the specifications following queries as to the corresponding alternative or special requirements of individuals. These specifications have been adopted entirely or substantially by several important roads, are recommended by the American Bridge Company, are acceptable to a large number of engineers and manufacturers, and will doubtless be formally adopted by other railroads and engineers. They have therefore been included in the preceding tabulations, and on account of their importance and up-to-date value the principal clauses applicable to the work here considered are here rearranged and presented.

General Specifications for Steel Railroad Bridges, 1906.

AMERICAN RAILWAY ENGINEERING AND MAINTENANCE OF WAY ASSOCIATION.

Recommended Practice in Contracting for Steel Railroad Bridges.

1. That it is preferable for railroads to furnish general detailed plans and specifications of structural work to bidders complete enough to show the exact character of the work; but if such plans cannot be furnished, the alternative to be full specifications similar to those on pages 6 to 25, accompanied by outline plans and all information concerning the work.

2. That it is preferable to invite bids on a pound price basis; and, if desired, alternate bids may be asked for the work, f.o.b. cars, and for the work erected. That a lump sum bid is inadmissible unless general detailed plans and specifications are furnished.

3. That it is preferable to invite bids for as large groups of bridges as can be defined consistently with the first recommendation, but when required to anticipate future requirements it is not necessary for the railroad to submit designs if the nature of the work is known to the bidder by reason of having previously done work for the railroad or if standard designs of similar structures are submitted to the bidders.

4. That wherever a bridge is to be erected on a line where traffic is to be maintained, it is recommended that the work be done by the railroad force; but on small railroads where suitably organized and equipped forces for such work may not be justified, the large bridges, and in some cases all bridges, may be erected by contract.

5. That it is preferable in all cases that the railroad company furnish and lay the floor timber.

Points to be Specifically Determined by Buyers when Soliciting Proposals for Steel Railroad Bridges.

When general detailed drawings are not furnished for the use of bidders specific answers should be given to the following points: —

Specify class of live load.

If bridge is on curve, specify speed and superelevation.

Specify the kind of masonry on which the bridge will rest.

Shall hinged bolsters be used on spans of 80 ft. and over?

Shall plate girders over 50 ft. long be built with camber?

Shall general reaming be done?

Shall edge planing be done? If so, specify on what members?

Shall all floor connection angles be milled after riveting?

Shall field connections be assembled at the shop?

Specify kind of oil or paint to be used.

PART FIRST — DESIGN.

I. GENERAL FEATURES.

Kind of Material. The material in the superstructure shall be structural steel, except rivets, and as may be otherwise specified.

Spacing Trusses. The width center to center of girders and trusses shall in no case be less than one-twentieth of the effec-

tive span, nor less than is necessary to prevent overturning under the assumed lateral loading.

Skew Bridges. Ends of deck plate girders and track stringers of skew bridges at abutments shall be square to the track, unless a ballasted floor is used.

Timber Floors. Wooden tie floors shall be secured to the stringers and shall be proportioned to carry the maximum wheel load, with 100 per cent impact, distributed over three ties, with fiber strain not to exceed 2,000 lbs. per sq. in. Ties shall not be less than 10 ft. in length. They shall be spaced with not more than 6-in. openings, and shall be secured against bunching.

II. LOADS.

Dead Load. The dead load shall consist of the estimated weight of the entire suspended structure. Timber shall be assumed to weigh $4\frac{1}{2}$ lbs. per ft. B. M., ballast 100 lbs. per cu. ft., and rails and fastenings 150 lbs. per linear ft. of track.

Moving Load. The live load for each track shall consist of two typical engines followed by a uniform load, according to Cooper's series, or a system of loading giving practically equivalent strains. The minimum loading to be Cooper's E-40, or two 50,000-lb. axle loads 7 ft. cc., the diagram that gives the larger strains to be used.

Heavier Loading. Heavier loadings shall be proportional to the same spacing.

Impact. The dynamic increment of the live load shall be added to the maximum computed live load strains, and shall be determined by the formula $I = S \frac{300}{L = 300}$,

where I = impact or dynamic increment to be added to live load strains.

S = computed maximum live load strain.

L = loaded length of track in feet producing the maximum strain in the member. For bridges carrying more than one track the aggregate length of all tracks producing the strain shall be used.

Impact shall not be added to strains produced by longitudinal, centrifugal, and lateral or wind forces.

Lateral Load. All spans shall be designed for a lateral force on the loaded chord of 200 lbs. per linear foot plus 10 per cent

of the specified train load on one track, and 200 lbs. per linear foot on the unloaded chord, these forces being considered as moving.

Centrifugal Force. Structures located on curves shall be designed for the centrifugal force of the live load acting at a height of 6 ft. above the rail, proper account being taken of the superelevation, the speed and superelevation to be determined by the engineer for each case.

III. UNIT STRAINS AND PROPORTION OF PARTS.

Unit Strains. All parts of structures shall be so proportioned that the sum of the maximum strains shall not exceed the following amounts in pounds per sq. in., except as modified in paragraphs 20 to 23 :

Tension.	Axial tension on net section . . .	16,000
Compression.	Axial compression on gross section . . . . . where l is the length of member in inches and r is the least radius of gyration in inches.	$16{,}000 - 70\frac{l}{r}$
Bending.	On extreme fibers of rolled shapes, built sections and girders ; net section .	16,000
	On extreme fibers of pins	24,000
Shearing.	Shop driven rivets and pins	12,000
	Field driven rivets and turned bolts . .	10,000
	Plate girder webs, gross section . . .	10,000
Bearing.	Shop driven rivets and pins	24,000
	Field driven rivets and turned bolts . .	20,000
	Granite masonry and Portland cement concrete	600
	Sandstone and limestone	400
	Expansion rollers, per linear inch . . where d is the diameter of the roller in inches.	$600\,d$

Alternate Strains. Members subject to alternate strains of tension and compression shall be proportioned for the strains giving the largest section. If the alternate strains occur in succession during the passage of one train, as in stiff counters, each strain shall be increased by 50 per cent of the smaller. The connections shall in all cases be proportioned for the sum of the strains.

Counter-strains. Wherever the live and dead load strains are of opposite character only 70 per cent of the dead loads train shall be considered as effective in counteracting the live load strain.

Axial and Bending Strains Combined. Members subject to both axial and bending strains shall be proportioned so that the combined fiber strains will not exceed the allowed axial strain.

Lateral and Other Strains Combined. For strains produced by longitudinal and lateral or wind forces combined with those from live and dead load and centrifugal forces, the unit strain may be increased 25 per cent over those given above; but the section shall not be less than required if the longitudinal and lateral or wind forces be neglected.

Net Section at Rivets. In proportioning tension members the diameter of the rivet holes shall be taken $\frac{1}{8}$ in. larger than the nominal diameter of the rivet.

Rivets. In proportioning rivets the nominal diameter of the rivet shall be used.

Proportioning Plate Girders. Plate girders shall be proportioned either by the moment of inertia of their net section, or by assuming that the flanges are concentrated at their centers of gravity, in which case one-eighth of the gross section of the web, if properly spliced, may be used as flange section.

Compression Flange. The gross section of the compression flanges of plate girders shall not be less than the gross section of the tension flanges, nor shall the strain per square inch in the compression flange of any beam or girder exceed $16{,}000 - 200\,\frac{l}{b}$, where l = unsupported distance and b = width of flange.

Flange Rivets. The flanges of plate girders shall be connected to the web with a sufficient number of rivets to transfer the total shear at any point in a distance equal to the effective depth of the girder at that point combined with any load that is applied directly on the flange. The wheel loads where the ties rest on the flanges shall be assumed to be distributed over three ties.

Depth Ratios. Trusses shall preferably have a depth of not less than one-tenth of the span. Plate girders and rolled beams used as girders shall preferably have a depth of not less than one-twelfth of the span. If shallower trusses, girders, or beams are used the section shall be increased so that the maximum deflection will not be greater than if the above limiting ratios had not been exceeded.

IV. DETAILS OF DESIGN.

General Requirements.

Compression Members. In compression members the metal shall be concentrated as much as possible in webs and flanges. The thickness of each web shall be not less than one-thirtieth of the distance between its connections to the flanges. Cover plates shall have a thickness not less than one-fortieth of the distance between rivet lines.

Minimum Angles. Flanges of girders and built members without cover plates shall have a minimum thickness of one-twelfth of the width of the outstanding leg.

Expansion. Provision for expansion to the extent of $\frac{1}{8}$ inch for each 10 feet shall be made for all bridge structures. Efficient means shall be provided to prevent excessive motion at any one point.

Expansion Bearings. Spans of 80 ft. and over resting on masonry shall have turned rollers or rockers at one end, and those of less length shall be arranged to slide on smooth surfaces.

Fixed Bearings. Movable bearings shall be designed to permit motion in one direction only. Fixed bearings shall be firmly anchored to the masonry.

Rollers. Expansion rollers shall be not less than 6 in. in diameter. They shall be coupled together with substantial side bars, which shall be so arranged that the rollers can be readily cleaned.

Bolsters. Bolsters or shoes shall be so constructed that the load will be distributed over the entire bearing. [Spans of 80 ft. or over shall have hinged bolsters at each end.]

Wall Plates. Wall plates may be cast or built up, and shall be so designed as to distribute the load uniformly over the entire bearing. They shall be secured against displacement.

Inclined Bearings. Bridges on an inclined grade without pin shoes shall have the sole plates beveled so that the masonry and expansion surfaces may be level.

Floor Systems.

Floorbeams. Floorbeams shall preferably be square to the girders. They shall be riveted directly to the girders or trusses or may be placed on top of deck bridges.

Stringers. Stringers shall preferably be riveted to the webs of all intermediate floorbeams by means of connection angles

not less than $\frac{7}{16}$ in. thick. Shelf angles or other supports provided to support the stringer during erection shall not be considered as carrying any of the reaction.

End Spacers for Stringers. Where end floorbeams cannot be used, stringers resting on masonry shall have cross frames near their ends. These frames shall be riveted to girder or truss shoe where practicable.

Plate Girders.

Camber. [If desired, plate girder spans over 50 ft. in length shall be built with camber at a rate of $\frac{1}{16}$ in. per 10 ft. of length.]

Top Flange Cover. Where flange plates are used, one cover plate of top flange shall extend the whole length of the girder.

Web Stiffeners. There shall be web stiffeners, generally in pairs, over bearings, at points of concentrated loading and at points required by the formula: $d = \frac{t}{40}(12{,}000 - S)$,

where d = clear distance, between stiffeners or flange angles,

t = thickness of web,

S = shear per sq. in.

The stiffeners at ends and at points of concentrated loads shall be proportioned by the formula of paragragh 16, the effective length being assumed as one-half the depth of girders. End stiffeners and those under concentrated loads shall be on fillers and have their outstanding legs as wide as the flange angles will allow and shall fit tightly against them. Intermediate stiffeners may be offset or on fillers and their outstanding legs shall be not less than one-thirtieth of the depth of girder plus 2 in.

Stays for Top Flanges. Through plate girders shall have their top flanges swayed at each end of every floorbeam, or in case of solid floors, at distances not exceeding 12 ft., by knee braces or gusset plates.

PART SECOND — MATERIALS AND WORKMANSHIP.

V. MATERIAL.

Process of Manufacture. Steel shall be made by the open-hearth process.

Schedule of Requirements. The chemical and physical properties shall conform to the following limits:

Elements Considered.	Structural Steel.	Rivet Steel.	Steel Castings.
Phosphorus, max. { Basic	0.04 per cent	0.04 per cent	0.05 per cent
Phosphorus, max. { Acid	0.06 "	0.04 "	0.08 "
Sulphur, maximum	0.05 "	0.04 "	0.05 "
Ultimate tensile strength.	Desired	Desired	Not less than
Pounds, per square inch	60,000	50,000	65,000
Elong., min. % in 8", Fig. 1	1,500,000	1,500,000	15 per cent
" " " 2", " 2	Ult. tensile strength 22	Ult. tensile strength	
Character of Fracture	Silky	Silky	Silky or fine granular
Cold Bends without Fracture	180° flat	180° flat	90° $d = 3t$

The yield point, as indicated by the drop of beam, shall be recorded in the test reports.

Allowable Variations. If the ultimate strength varies more than 4,000 lbs. from that desired, a retest shall be made on the same gage, which, to be acceptable, shall be within 5,000 lbs. of the desired ultimate.

Chemical Analyses. Chemical determinations of the percentages of carbon, phosphorus, sulphur, and manganese shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector. Check analyses shall be made from finished material, if called for by the purchaser, in which case an excess of 25 per cent above the required limits will be allowed.

Modifications in Elongation. For material less than $\frac{5}{16}$-in. and more than $\frac{3}{4}$-in. in thickness the following modifications will be allowed in the requirements for elongation:

For each $\frac{1}{16}$-in. in thickness below $\frac{1}{2}$-in., a deduction of $2\frac{1}{2}$ will be allowed from the specified percentage.

For each $\frac{1}{8}$-in. in thickness above $\frac{3}{4}$-in., a deduction of 1 will be allowed from the specified percentage.

Bending Angles. Angles $\frac{3}{4}$-in. and less in thickness shall open flat, and angles $\frac{1}{2}$-in. and less in thickness shall bend shut, cold, under blows of a hammer, without sign of fracture. This test will be made only when required by the inspector.

Rivet Holes. When general reaming is not required the diameter of the punch shall not be more than $\frac{1}{16}$-in. greater than the diameter of the rivet; nor the diameter of the die more than $\frac{1}{8}$-in. greater than the diameter of the punch. Material more than $\frac{3}{4}$-in. thick shall be sub-punched and reamed or drilled from the solid.

Punching. All punching shall be accurately done. Drifting to enlarge unfair holes will not be allowed. If the holes must be enlarged to admit the rivet, they shall be reamed. Poor matching of holes will be cause for rejection.

Sub-punching and Reaming. Where reaming is required, the punch used shall have a diameter not less than $\frac{3}{16}$-in. smaller than the nominal diameter of the rivet. Holes shall then be reamed to a diameter not more than $\frac{1}{16}$-in. larger than the nominal diameter of the rivet. All reaming shall be done with twist drills.

Webb Stiffeners. Stiffeners shall fit neatly between flanges of girders. Where tight fits are called for, the ends of the stiffeners shall be faced and shall be brought to a true contact bearing with the flange angles.

Splice Plates and Fillers. Web splice plates and fillers under stiffeners shall be cut to fit within $\frac{1}{8}$-in. of flange angles.

Web Plates. Web plates of girders, which have no cover plates, shall be flush with the backs of angles or project above the same not more than $\frac{1}{8}$ in., unless otherwise called for. When web plates are spliced, not more than $\frac{1}{4}$ in. clearance between ends of plates will be allowed.

Connection Angles. Connection angles for floorbeams and stringers shall be flush with each other and correct as to position and length of girder. In case milling [of all such angles] is needed or is required after riveting, the removal of more than $\frac{1}{16}$-in. from their thickness will be cause for rejection.

Field Connections. Holes for floorbeam and stringer connections shall be sub-punched and reamed according to paragraph 117, to a steel templet one inch thick. [If required, all other field connections, except those for laterals and sway bracing, shall be assembled in the shop and the unfair holes

reamed; and when so reamed, the pieces shall be match-marked before being taken apart.]

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A. F. Robinson, Bridge Engineer, Santa Fe Railway System, Chicago, Ill.

C. C. Schneider, Consulting Engineer, American Bridge Company, Philadelphia, Pa.

J. R. Worcester, Consulting Engineer, Boston, Mass.

Committee.

Tables Prepared by American Bridge Company.

Table 20. — Maximum Moments M, Endshears S, and Floorbeam Reactions R per track, produced by Cooper's Standard Loading E 40, on Spans from 10 to 250 feet.

Span L in ft.	Max. Mom. M in ft. lbs.	Max. Endshear S in lbs.	Max Flbm. Reac. R in lbs.	Equivalent Uniform Load.		
				M	S	R
10.....	112,500	60,000	82,000	9,000	12,000	8,200
11.....	140,800	64,000	87,600	9,310	11,640	7,960
12.....	169,100	68,000	93,200	9,340	11,330	7,830
13.....	197,400	72,000	98,800	9,340	11,080	7,600
14.....	225,700	76,000	104,400	9,210	10,860	7,460
15.....	254,000	80,000	110,000	9,030	10,670	7,330
16.....	283,200	84,000	114,000	8,850	10,500	7,120
17.....	312,400	88,000	118,000	8,650	10,350	6,940
18.....	341,600	92,000	122,000	8,430	10,240	6,780
19.....	370,800	96,000	126,000	8,220	10,100	6,630
20.....	400,000	100,000	130,000	8,000	10,000	6,500
21.....	443,300	102,700	134,200	8,040	9,780	6,390
22.....	486,600	105,400	138,400	8,040	9,580	6,290
23.....	529,900	108,100	142,600	8,010	9,400	6,200
24.....	573,200	110,800	146,800	7,960	9,230	6,120
25.....	616,500	113,500	151,000	7,890	9,080	6,040
26.....	657,400	116,100	155,100	7,780	8,930	5,970
27.....	698,200	118,700	159,200	7,660	8,790	5,900
28.....	739,000	121,300	163,300	7,540	8,660	5,830
29.....	780,000	123,900	167,400	7,420	8,540	5,770
30.....	820,800	126,500	171,500	7,300	8,430	5,720
31.....	867,100	128,800	176,200	7,220	8,320	5,680
32.....	913,500	131,100	180,900	7,140	8,190	5,650
33.....	959,800	133,400	185,600	7,050	8,080	5,620
34	1,006,200	135,700	190,300	6,960	7,980	5,600
35	1,052,500	138,000	195,000	6,870	7,890	5,570
36	1,104,500	140,700	199,200	6,820	7,820	5,530
37	1,156,500	143,400	203,400	6,760	7,750	5,500
38	1,208,500	146,100	207,600	6,700	7,690	5,460
39	1,260,500	148,800	211,800	6,630	7,630	5,430
40	1,312,500	151,500	216,000	6,560	7,570	5,400
42	1,439,000	156,500	224,400	6,530	7,450	5,340
44	1,565,000	161,400	233,000	6,470	7,340	5,300

TABLE 21 — MAXIMUM MOMENTS M, ENDSHEARS S, AND FLOORBEAM REACTIONS R PER TRACK, PRODUCED BY COOPER'S STANDARD LOADING E 40, ON SPANS FROM 10 TO 125 FEET.

Span L in ft.	Max. Mom. M in ft. lbs.	Max Endshear S in lbs.	Max Flbm. Reac. R in lbs.	Equivalent Uniform Load.		
				M	S	R
46	1,674,000	166,500	241,200	6,330	7,240	5,240
48	1,791,100	171,400	249,600	6,220	7,140	5,200
50	1,908,300	176,400	257,000	6,110	7,060	5,140
52	2,040,100	180,300	266,600	6,040	6,940	5,130
54	2,172,000	184,200	276,200	5,960	6,820	5,120
56	2,305,000	188,100	285,800	5,880	6,720	5,110
58	2,438,000	192,000	295,400	5,800	6,620	5,090
60	2,580,000	196,000	305,000	5,730	6,530	5,080
62	2,735,000	201,200	314,800	5,690	6,490	5,080
64	2,920,000	206,400	324,600	5,700	6,450	5,070
66	3,060,000	213,000	334,400	5,620	6,450	5,070
68	3,216,000	216,800	344,200	5,560	6,380	5,060
70	3,372,000	222,000	354,000	5,510	6,340	5,060
72	3,556,000	227,600	362,400	5,490	6,320	5,030
74	3 740,000	233,200	370,800	5,460	6,300	5,010
76	3,924,000	238,800	379,200	5,440	6,290	4,990
78	4,122,000	244,400	387,600	5,420	6,270	4,970
80	4,320,000	250,000	396,000	5,400	6,250	4,950
82	4,515,000	255,200	404,200	5,370	6,230	4,930
84	4,710,000	260,400	412,400	5,340	6,200	4,910
86	4,905,000	265,600	420,600	5,310	6,180	4,890
88	5,100,000	270,800	428,800	5,270	6,150	4,870
90	5,320,000	276,000	437,000	5,250	6,130	4,860
92	5,560,000	281,000	444,400	5,250	6,110	4,830
94	5,752,000	286,100	451,800	5,210	6,090	4,810
96	5,960,000	291,000	459,200	5,170	6,060	4,780
98	6,180,000	296,100	466,600	5,150	6,040	4,760
100	6,436,000	301,200	474,000	5,140	6,020	4,704
125	9,960,000	360,900		5,100	5,770	

TABLE 22. — SHEARING AND BEARING VALUE OF RIVETS IN POUNDS.

Diam. of Rivet Inches.		Area in Square Inches.	Single Shear at 11,000 Lbs.	Bearing Value for Different Thicknesses of Plate in Inches at 22,000 Pounds per Square Inch.												
Fraction.	Decimal.			$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
$\frac{3}{8}$	.375	.1104	1210	2060	2580	3090										
$\frac{1}{2}$	.500	.1963	2160	2750	3440	4130	4820	5500								
$\frac{5}{8}$	.625	.3068	3370	3440	4300	5160	6020	6880	7740	8600						
$\frac{3}{4}$	.750	.4418	4860	4130	5160	6190	7220	8250	9280	10,320	11,340	12,380				
$\frac{7}{8}$	.875	.6013	6610	4810	6020	7220	8430	9630	10,840	12,040	13,240	14,440	15,640	16,840	18,050	
1	1.000	.7854	8640	5500	6880	8250	9630	11,000	12,380	13,750	15,130	16,500	17,880	19,250	20,630	22,00

All Bearing Values above or to right of upper Zigzag Lines are greater than Double Shear.
Values below or to left of lower Zigzag Lines are less than Single Shear.

TABLE 23. — COEFFICIENTS OF IMPACT, I.

Computed from formula $I = S\left(\frac{300}{L+300}\right)$ where I is impact to be added to live load strain, S is calculated maximum live load strain, and L is length in feet of loaded distance which produces maximum strain.

L.	$\frac{300}{L+300}$	L.	$\frac{300}{L+300}$	L.	$\frac{300}{L+300}$	L.	$\frac{300}{L+300}$
5	0.984	31	0.906	57	0.840	83	0.783
6	0.980	32	0.904	58	0.838	84	0.781
7	0.977	33	0.901	59	0.836	85	0.779
8	0.974	34	0.898	60	0.833	86	0.777
9	0.971	35	0.896	61	0.831	87	0.775
10	0.968	36	0.893	62	0.829	88	0.773
11	0.965	37	0.890	63	0.826	89	0.771
12	0.962	38	0.888	64	0.824	90	0.769
13	0.958	39	0.885	65	0.822	91	0.767
14	0.955	40	0.882	66	0.820	92	0.765
15	0.952	41	0.880	67	0.817	93	0.763
16	0.949	42	0.877	68	0.815	94	0.761
17	0.946	43	0.875	69	0.813	95	0.759
18	0.943	44	0.872	70	0.811	96	0.758
19	0.940	45	0.870	71	0.809	97	0.756
20	0.937	46	0.867	72	0.806	98	0.754
21	0.935	47	0.865	73	0.804	99	0.752
22	0.932	48	0.862	74	0.802	100	0.750
23	0.929	49	0.860	75	0.800	105	0.741
24	0.926	50	0.857	76	0.798	110	0.732
25	0.923	51	0.855	77	0.796	115	0.725
26	0.920	52	0.852	78	0.794	120	0.714
27	0.917	53	0.850	79	0.793	125	0.706
28	0.915	54	0.847	80	0.789	130	0.698
29	0.912	55	0.845	81	0.787	135	0.690
30	0.909	56	0.843	82	0.785	140	0.682

Atchison, Topeka, and Sante Fe Railroad (No. 2).

Elevation of Rails. When girders are on curves, the super-elevation for outer rail will be made in the pedestal stones.

Maximum Unit Tensile Stresses for dead and live loads respectively. Bottom flanges 18,000 and 9,000 lb. Lateral bracing 15,000 and 8,000 lb.

Transverse Diagonal Bracing in deck spans shall be strong enough to cause the two girders to act as a unit in overturning.

Baltimore and Ohio Railroad (No. 3).

Tracings made on dull side of cloth in uniform sheets 24 × 36 in. for detail drawings and 11½ × 18 in. for strain sheets.

On Curves the center line of bridge is parallel to chord of curve. On a curve, both girders are to be alike and are to

be calculated for a proportion of the live load $= P(m + b) \div 2b$, where m is the center ordinate to the curve, b is the distance between centers of girders, and P is live load at panel point.

Skew Spans have the ends of girders extended over abutments to finish square with bridge axis when practicable.

Wall Plates may be cast or riveted with minimum thickness of $\frac{3}{4}$ in. and must be designed to distribute the load equally over entire bearing. Where wall plates do not have full masonry bearing efficiency shall be secured with rust cement, Portland cement mortar, or by bedding the plate on a thin layer of dry neat Portland cement on masonry. Stone bolts shall penetrate the masonry at least 9 in. and be set in Portland cement.

Choctaw, Oklahoma, and Gulf Railroad (No. 4).

Hand Driven Rivets have 80 per cent the efficiency of machine driven rivets and floorbeam connection rivets must have 25 per cent added to live load for impact.

Summation of Stresses. Permissible unit strains due to wind stresses, $\frac{2}{3}$ those allowed for dead load. Shearing and bearing values of rivets in wind stress connections 50 per cent greater than those for live and dead loads. Stresses from wind, tractive force, or centrifugal force shall not be added to the live and dead load stresses unless they amount to 50 per cent of them; when added the unit strain allowed shall be $1\frac{1}{2}$ dead load unit strain.

Reaming. Rivet hole reamers must not be tapered or operated with a flexible shaft. The ends of floorbeams must be faced to length. Floorbeam connection rivet holes must be reamed through metal templates not less than $1\frac{1}{4}$ in. thick.

Elastic Limit. In tension tests the "Yield Point" must be determined by the drop of the beam or halt in the gauge of the testing machine, and subject to check by direct measurement with dividers.

Overweight Allowed on Rectangular plates rolled to gauge varies from $3\frac{1}{2}$ per cent for plates over $\frac{5}{8}$ in. thick and up to 75 in. wide, to 18 per cent for $\frac{1}{4}$ in. plates over 100 in. wide.

Boston and Maine Railroad (No. 6).

Unit Tension Strains. Unit strains will be based upon the ratio of dead load to total load, modified as specified below.

The following table gives general tension units:

TABLE 24.

Ratio, R.	Unit Strain, A.	Ratio, R.	Unit Strain, A.	Ratio, R.	Unit Strain, A.	Ratio, R.	Unit Strain, A.	Ratio, R.	Unit Strain, A.
— 1 0	5,000	— .5	5,800	— .1	7,850	.3	10,650	.7	13,380
— .8	5,170	— .4	6,200	.0	8,500	.4	11,355	.8	14,000
— .7	5,300	— .3	6,660	.1	9,225	.5	12,030	.9	14,560
— .6	5,530	— .2	7,210	.2	9,940	.6	12,720	1.0	15,000

For members subjected to one kind of strain only,

$$R = \frac{\text{Dead load}}{\text{Total load}}.$$

Alternate Strains. When the compression is numerically the greater, the tension unit will remain at 5,000. The compression unit shall be as specified below, and that one used which gives the larger gross section.

Flange Stresses. In tension flanges the unit of stress shall be $T = a[\frac{1}{2} + (5d \div l)]$ where a is taken from accompanying table, d is strain depth, and l is effective length of girder; but stress shall never exceed a. In compression flanges the above unit stress shall be $(T \div 10{,}000) \times [9{,}000 \div \{l + (l^2 \div 5{,}000b^2)\}]$ where l is the unsupported length and b the width of the flange. Where the girder is riveted at both ends to the webs of other girders the unsupported length may be taken as $\frac{3}{4}$ the actual length of the top flange.

Web Shear. The shear on webs, per square in. of vertical section, shall not exceed $(a \div 10{,}000) \times \{12{,}000 \div [l + (d^2 \div 2{,}500\ t^2)]\}$ where d is depth between flange angles or clear distance between uprights, and t is the thickness of the web.

Rivets. Unit stresses for rivets shall be for machine driven rivets, single action $a \times 1.2500$, hand driven, $a \times 0.9375$. Machine driven enclosed bearing, $a \times 1.5625$, hand driven, $a \times 1.1720$. In single shear, for machine or hand driven, $a \times 0.75$, double shear, $a \times 1.5$. Hand driven rivets must be iron or basic steel below 52,000 lb. ultimate. In case of rivets transmitting strain between parts separated by intervening metal, from $1\frac{1}{2}$ to 2 times as many rivets shall be provided as the strain units would otherwise call for.

Trestles on straight lines shall be computed for a lateral force, applied at the rail level, of 500 lb. per lin. ft. If over 30 ft. high an additional force of 30 lb. per square ft. on the surface

of one girder and tower shall be considered. When unloaded they must be stable under a lateral force of 50 lb. per square ft. on $1\frac{1}{2}$ times the above surface. On curves centrifugal force must be provided for in addition to the above lateral force.

Longitudinal Bracing shall be provided to resist a force equal to 0.2 the live load between centers of braced towers.

In Bridges on Curves the centrifugal force per ft. of span shall be computed by the following formula:

$$\text{Centrifugal force pr. ft.} = \frac{110 \times \text{live load pr. ft. on all tracks}}{\text{radius of curve}}.$$

The chord (flange) strain from the centrifugal force shall be added to that from the vertical loading, but that from the other lateral load specified above need not be considered, unless the flange strain from the lateral load amounts to 30 per cent of that from the vertical load, in which case the section shall be adjusted so that the unit strain from the combined load shall not exceed that prescribed below by more than 30 per cent.

Bearing per lin. in. of rollers, 350 × diam. for iron; 470 × diam. for medium steel; 540 × diam. for hard steel. Wall plates will be preferably of cast iron of a thickness at least $\frac{1}{10}$ the largest dimension of the shoe plate resting on them, and will have anchor bolts secured to the masonry with sulphur or neat Portland cement.

In Double Track bridges the unit stresses may be increased $7\frac{1}{2}$ per cent and 10 per cent respectively when there are two and three girders.

Post Supports. Two girders supported independently on the same post shall have pin bearings or be headed into the post with faced ends.

Canadian Pacific Railway (No. 7).

Field Driven Rivets shall be iron.

Paint. Shop paint shall be made with one gallon pure boiled linseed oil, 30 lb. pure red lead, and not more than 12 oz. lamp black. The first coat of field paint shall be the same except that it shall have 25 lb. of red lead.

Chicago Great Western Railway (No. 8).

Plates up to 36 in. wide must be universal mill rolled.

Tower Spans for high trestles shall be not less than 30 per cent of the height nor less than 30 ft. in length.

Riveted Pedestals for spans of 40 ft. and over must have the planed bearings between the base and web plates, and must be joined by angles having two rows of rivets in the vertical flanges.

Anchor Bolts must extend 30 in. in the masonry and must have a diameter of $1\frac{1}{4}$ in. for 40-ft. spans and $\frac{1}{8}$ in. more for every additional 20 ft. of span.

Cleveland, Cincinnati, Chicago, and St. Louis Railway (No. 11).

On Curves girders will be located to receive equal loads, thus usually making the distance from the center of the track to the axis of the bridge from $\frac{1}{3}$ to $\frac{1}{2}$ the middle ordinate.

Center Girders. In double track, spans with three girders will be proportioned for 90 per cent of the live load coming to them from both tracks; when there are only two girders, each will be proportioned for 90 per cent of the load on one track.

Web Flange Rivets shall be spaced by the formula, $P = rd \div s$, where P is the pitch, r is the value of one rivet, and d is the depth, center to center of rivet lines, and s is the shear at that point. No rivet grip shall be more than 4 in. Flange angles 8 in. wide shall have three lines of rivets.

The Fiber shall run lengthwise of the girder in web plates.

Bracing. There shall be top and bottom laterals for deck spans of over 35 ft. Deck spans shall have stiff cross-frames at the ends and at intervals of 20 ft. proportioned to resist unequal loading of the girders, those at the ends having the same strength as the top lateral bracing.

Anchor Bolts at expansion ends shall have turned shoulders for their nut bearings.

Shop Assembling. All plate girders must be temporarily assembled before shipment.

Standard Specifications (No. 10).

All tests and inspections shall be made at place of manufacture prior to shipment.

The tensile strength, limit of elasticity, and ductility shall be determined from a standard test piece cut from the finished material. The standard shape of the test piece for sheared plates shall be as shown. On tests cut from other material the test piece may be either the same as for plates or it may be planed or turned parallel throughout its entire length. The elongation shall be measured on an original length of eight

inches, except in rounds of $\frac{5}{8}$ inch or less in diameter, in which case the elongation shall be measured in a length equal to eight times the diameter of section tested. Four test pieces shall be taken from each melt of finished material; two for tension and two for bending.

Material which is to be used without annealing or further treatment is to be tested in the condition in which it comes from the rolls. When material is to be annealed or otherwise treated before use, the specimen representing such material is to be similarly treated before testing.

Every finished piece of steel shall be stamped with the melt number.

All plates shall be free from surface defects and have a workmanlike finish.

Theodore Cooper (No. 12).

Adjacent Spans. When two spans rest on the same masonry their bearings must be tied rigidly together or must have a continuous plate under them.

Pedestals must be made with riveted plates and angles, with the bearings between them riveted and connected by angles not less than $\frac{3}{4}$ in. thick with two rows of rivets through the vertical flange. When the size of the pedestal permits, the vertical webs must be rigidly connected transversely.

These specifications are accompanied by tables of weights and dimensions and driver loads for recent heavy passenger and freight engines, maximum moments, end shears, floorbeam reactions, and equivalent uniform loadings for girders of 10 to 125 ft. span and for maximum moments, end shears and equivalent uniform loads produced by special loads on two axles, and by a table of axle loads and spaces for standard locomotives and train loads designated as E 27, E 30, E 35, E 40, and E 50 where the numbers 27 to 50 indicate the number of thousand pounds on each of four driver axles 5 ft. apart. In these diagrams the axle spacing is assumed the same for all the different weights, and all have a total wheel base of 48 ft. including four driver axles, four tender axles, and one pilot axle spaced 8 ft. from the rear tender axle of the preceeding of two coupled locomotives. Mr. Cooper notes that the heaviest locomotives in use on United States railroads in 1900 have the same effect on bridges as the typical loading marked E 35 to E 50, and that coal cars with a total load of 146,000 lb. on two axles produce strains equiva-

TABLE 25.—COOPER'S STANDARD LOADING.

DISTANCES IN FEET.

Class	Uniform Load.	5000 lbs. per lin. ft.	4500 lbs. per lin. ft.	4000 lbs. per lin. ft.	3000 lbs. per lin. ft.	.....
E 50		32500	29250	26000	19500	
		32500	29250	26000	19500	
		32500	29250	26000	19500	
		32500	29250	26000	19500	
		50000	45000	40000	30000	
		50000	45000	40000	30000	
		50000	45000	40000	30000	
		50000	45000	40000	30000	
		25000	22500	20000	15000	
E 45		32500	29250	26000	19500	
		32500	29250	26000	19500	
		32500	29250	26000	19500	
		32500	29250	26000	19500	
		50000	45000	40000	30000	
		50000	45000	40000	30000	
		50000	45000	40000	30000	
		50000	45000	40000	30000	
		25000	22500	20000	15000	
E 40						
E 30						
E . . .						

lent to those of E 33, and that for any United States railroad with a future E 35 train load should be the minimum adopted.

The Illinois Central Railroad Company. (13)

Wind Pressure shall be assumed acting in either direction horizontally :

On the loaded structure at 30 lb. per square ft., on the exposed surface of all trusses and the floor system, as seen in elevation, in addition to a train surface of 10 ft. average height, beginning 2 ft. 6 in. above base of rail, moving across the structure.

On the unloaded structure at 50 lb. per square ft. on the exposed surface of all trusses and the floor system as seen in elevation.

Working Stress. The greatest working stresses, in lbs. per square in. shall be in tension ;

Plates or shapes, net section . . . . . $8{,}000\left(1+\frac{\text{Min.}}{\text{Max.}}\right)$

For combined live, dead, wind, and centrifugal stresses, increase the preceding unit stress 30 per cent above live and dead load unit stresses.

Girder Spacing. Girders of deck-plate spans shall be spaced as follows, with necessary allowance on curves :

For lengths 60 ft. and under, center to center of girders, 7 ft.

For lengths from 60 ft. to 80 ft., center to center of girders, 8 ft.

For lengths from 80 ft. to 100 ft., center to center of girders, 9 ft.

Friction Rollers. All bridges over 75 ft. in span shall have at one end segmental steel friction rollers not less than 6 in. in diameter, running between parallel planed surfaces ; but cylindrical rollers 4 in. in diameter may be used when desired.

Fixed End. Where practicable the fixed end of a span must be as follows — letters indicating order of importance :

(*a*) At the end connecting with trestle approach.

(*b*) At one abutment.

(*c*) At the lowest pier.

(*d*) When the structure is on a grade, in the absence of other governing conditions, at the down grade end.

(*e*) In structures consisting of several spans, anchorages to be arranged so that no pier shall carry the fixed end of more than one span.

Bed Plate. When practicable, adjacent ends of consecutive spans shall have a common bed plate. When the height of bed plate exceeds 6 in., separators must be introduced or other provision made to insure horizontal stiffness.

Test. The completed bridge when tested by a train moving at the rate of 60 miles an hour shall not deflect more than one fifteen-hundredth of its span, and must return to its original camber after the passage of the train.

King Bridge Company (No. 14).

Blanks are provided for the principal dimensions of both substructure and superstructure.

Unit Working Stresses, S, for plates and shapes, are functions of the maximum and minimum stresses M and m respectively as derived by the formulas for direct and live and dead loads, of $S = 8{,}500\ [1 + (m - M)]$ for soft steel, and $S = 9{,}500\ [1 + (m - M)\ ]$ for medium steel. Unit stresses for combined wind, centrifugal, and momentum forces are 15,000 lb. and 17,000 lb. for soft and medium steel respectively. The unit stresses for rigid bracing subjected to alternate stresses is $S = C \div [1 + (L^2 \div 36{,}000\ R^2)\ ]$ where L is length, R is radius of gyration, and C is a constant of 10,000 for soft steel and 12,000 for medium steel.

New York Central and Hudson River Railroad (No. 15).

Floors shall generally consist of beams, stringers, and wooden ties, for through spans, the depths of floorbeams and stringers being usually not less than $\frac{1}{8}$ their length. Floors are designed with special reference to stiffness and where they must be shallow are usually solid steel construction. Solid floors are preferably made with rolled I-beams with plates not less than $\frac{7}{16}$ in. thick riveted to each beam. When trough floors are necessary on account of limited clearance they must be rectangular and built of plates and angles unless otherwise specified.

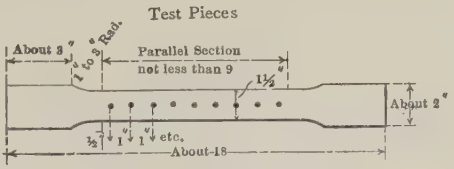


Fig. 1.

Deflection. When the depth of a girder is less than $\frac{1}{12}$ the span, they must have web and flange material increased to

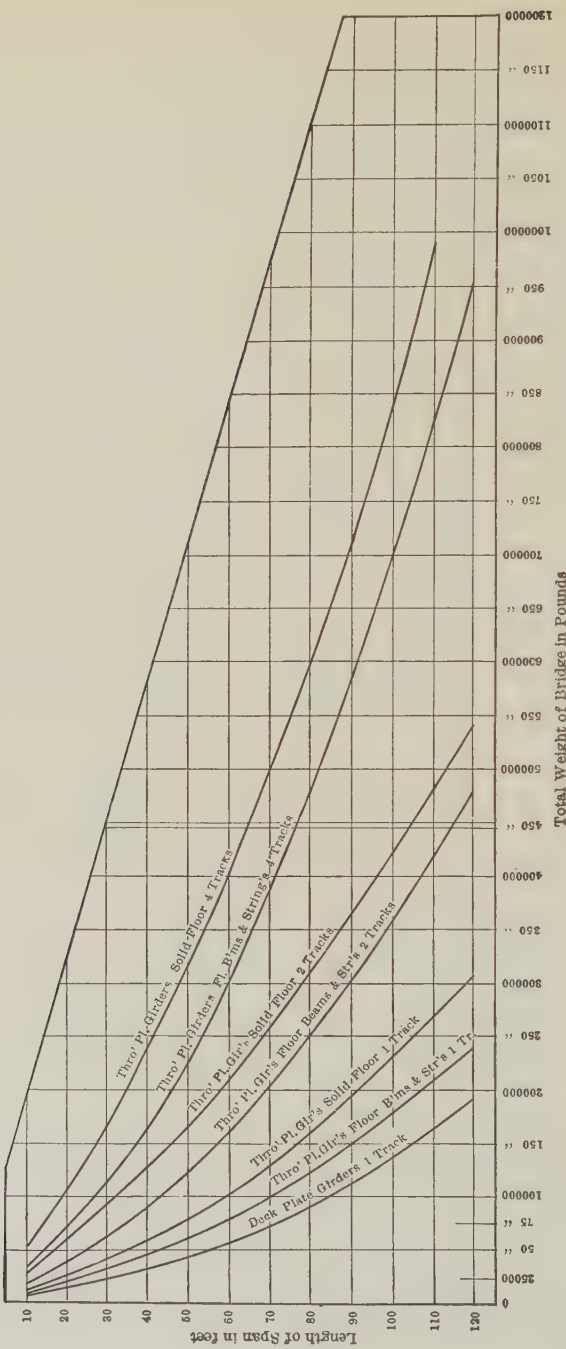

Thro' Pl.Girders Solid Floor 4 Tracks
Thro' Pl.Girders Pl. B'ms & Strings 4 Tracks
Thro' Pl.Gir's Solid Floor 2 Tracks
Thro' Pl. Gir's Floor Beams & Str's 2 Tracks
Thro' Pl. Gir's Solid Floor 1 Track
Thro' Pl. Gir's Floor B'ms & Str's 1 Tr
Thro' Pl.Gir's
Deck Plate Girders 1 Track
Length of Span in feet
Total Weight of Bridge in Pounds

Fig. 2.

diminish the deflection to that which would be given by a girder with a depth $\frac{1}{12}$ of its span.

Connections. Through span floorbeams must be connected to the girders with gusset plates not less than 16 in. wide, forming continuous portions of the floorbeam webs and riveted between the flange angles. The gusset plates shall be stiffened by a pair of $3\frac{1}{3} \times 3\frac{1}{2}$-in. flange angles on the inclined edge.

Deck Spans of less than 75 ft. shall be shipped riveted up complete.

Viaduct trestle bents shall preferably be united in pairs to form well braced towers and each bent shall have two columns

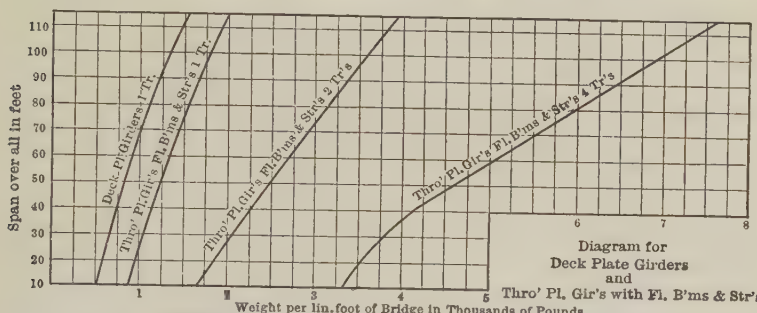


Diagram for Deck Plate Girders and Thro' Pl. Gir's with Fl. B'ms & Str's.

Estimated Weights of Floor Beams and Stringers including Laterals'

Floor carrying one Track					Floor carrying two Tracks				
Dep. Fl. B'ms	Len. Fl. B'ms	Dep. Str'gs	Len. Str'gs	Wt. per lin. ft. 1 Tr.	Dep. Fl. B'ms	Len. Fl. B'ms	Dep. Str'gs	Len. Str'gs	Wt. per lin. ft. 2 Tr.
1′-8⅜″	14′-0″	1′-1⅛″	7′-8″	465 lbs.	2′-10½″	27′-0″	2′-3⅜″	12′-5½″	1540 lbs.
2-0⅛	13-0	1-9	13-1	555 "	3-0	26-9	2-4⅜	12-1¾	1560 "
2-3⅛	14-0	1-7⅝	10-10	465 "	3-1¾	27-3	2-6⅛	13-5½	1565 "
2-3⅛	14-0	1-9⅛	14-10	498 "	3-5⅝	29-0	2-8⅛	15-9⅛	1250 "
2-6⅛	15-0	2-0⅛	14-9	518 "	4-0⅛	28-9	3-5⅛	17-8	1430 "
2-7⅛	15-0	2-0⅛	13-0	498 "					
2-9⅛	16-0	2-2⅛	15-10⅛	484 "					

Fig. 3.

battered transversely not less than 1 : 6 for single track and 1 : 8 for double track structures.

Bracing. In deck spans of less than 35 ft. the bottom laterals may be omitted. All deck spans shall have cross-frames at each end and spaced not more than 20 ft. intermediately. No lateral, longitudinal, nor sway brace angle shall be less than $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$-in. and all angles shall be connected with plates at their intersection and have at least four rivets in each connection.

Unit Bearing Stresses shall be 5,000 lb. for phosphor bronze, 250 lb. live and 500 lb. dead loads for stone masonry and 200 lb. live and 400 lb. dead for concrete masonry. The allowed bearing pressure per lineal inch on rollers shall not exceed $400\,d$, where d equals the diameter of the roller.

Roller Bearings must not hold water and must be accessible for cleaning and oiling the rollers.

Rivets. No $\frac{7}{8}$-in. or 1-in. rivet shall have a grip of more than 5 in.

Rivet bearing shall not exceed $1\frac{1}{2}$ times allowed tensile strain. Rivet shear shall not exceed $\frac{3}{4}$ unit tension stress.

All field rivet holes except for lateral, longitudinal, and field

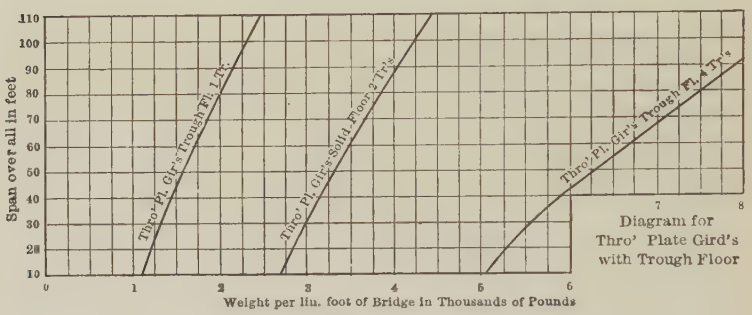


Fig. 4.

sway bracing must be drilled to metal templates or reamed while the connected members are assembled. Rivets more than $\frac{7}{8}$ in. diameter must not be hand driven. Bolts used in place of field rivets must be turned to a driving fit in drilled or reamed holes.

The specifications are accompanied by the table of locomotive and train loadings here given, by a table of maximum bending

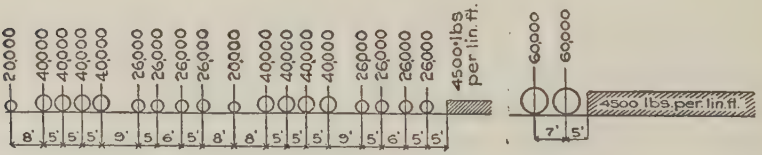


Fig. 5.

moments, end shears, and floorbeam concentrations for spans of 8 ft. to 100 ft. and by the accompanying diagrams of weights and by the details of spans shown in Chapter VII.

Floor Connections. Ends of solid floor sections connected to girders must come perfectly true and in line. If this cannot be secured otherwise the ends must be faced. The clearance with webs, stiffener angles, etc., must not exceed $\frac{1}{8}$ in. and must be so close that the asphaltum covering cannot run through.

Consolidation Engine

10.0	30.0	50.0	70.0	90.0	103.0	116.0	129.0	142.0	152.0	172.0	192.0	212.0	232.0	245.0	258.0	271.0	284.0	1
284.0	274.0	254.0	234.0	214.0	194.0	181.0	168.0	155.0	142.0	132.0	112.0	92.0	72.0	52.0	39.0	26.0	13.0	2
	8.0	13.0	18.0	23.0	32.0	37.0	43.0	48.0	56.0	64.0	83.0	74.0	79.0	88.0	93.0	99.0	104.0	109.0	3
	109.0	101.0	36.0	91.0	86.0	77.0	72.0	66.0	61.0	53.0	45.0	40.0	35.0	30.0	21.0	16.0	10.0	5.0	4
	8'	5'	5'	5'	9'	5'	6'	5'	8'	8'	5'	5'	5'	9'	5'	6'	5'	5'	
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	

2250# per lin. ft

16364.0	15274.0	13254.0	11334.0	9514.0	7794.0	6793.0	5857.0	4999.0	4206.0	3676.0	2776.0	1976.0	1276.0	676.0	403.0	195.0	65.0	5
14944.0	13904.0	11984.0	10164.0	8444.0	6824.0	5888.0	5017.0	4224.0	3496.0	3016.0	2216.0	1516.0	916.0	416.0	208.0	65.0		6
13589.0	12599.0	10779.0	9059.0	7439.0	5919.0	5048.0	4242.0	3514.0	2851.0	2421.0	1721.0	1121.0	621.0	221.0	78.0		65.0	7
10816.0	9936.0	8336.0	6836.0	5436.0	4136.0	3408.0	2745.0	2160.0	1640.0	1370.0	840.0	460.0	180.0			78.0	221.0	8
8728.0	7938.0	6518.0	5198.0	3978.0	2858.0	2247.0	1701.0	1233.0	830.0	600.0	300.0	100.0			65.0	208.0	416.0	9
7668.0	6928.0	5608.0	4388.0	3268.0	2248.0	1702.0	1221.0	818.0	480.0	300.0	100.0		100.0	282.0	529.0	854.0	1244.0	10
6708.0	6018.0	4798.0	3678.0	2658.0	1738.0	1257.0	841.0	503.0	230.0	100.0		100.0	300.0	547.0	859.0	1249.0	1704.0	11
5848.0	5208.0	4088.0	3068.0	2148.0	1328.0	912.0	561.0	288.0	80.0		100.0	300.0	600.0	912.0	1289.0	1744.0	2264.0	12
4632.0	4072.0	3112.0	2252.0	1492.0	832.0	520.0	273.0	104.0		160.0	420.0	780.0	1240.0	1656.0	2137.0	2696.0	3320.0	13

Line 1 – Summation of loads from left end. 2 – Summation of loads from right end 3 – Summation of distances from left end. 4 – Summation of distances from right end. 5 – Moments about end of uniform load 6 to 14 – Moments about wheels 18 to 10 respectively. Figures to left of heavy line are moments of loads to left of wheel indicated. Figures to right of heavy line are moments of loads to right of wheel indicated.

Special Loading

30.0	60.0		71.25	82.50	93.75	105.0	116.25	127.50	138.75	150.0	161.25	172.50	183.75	195.0	206.25	217.50	228.75	240.0	251.25	262.50	273.75	15
7	12	17	22	27	32	37	42	47	52	57	62	67	72	77	82	87	92	97	102	107	.	16
7'	5'	5'	5'	5'	5'	5'	5'	5'	5'	5'	5'	5'	5'	5'	5'	5'	5'	5'	5'	5'		
30000#	30000#	2250# per lin. ft																						
210.0	510.0	838.1	1222.5	1663.1	2160.0	2713.1	3322.5	3988.1	4710.0	5488.1	6322.5	7213.1	8160.0	9163.1	10222.5	11338.1	12510.0	13738.1	15022.5	16363.1	17
12908.1	11221.9	10153.1	9112.5	8128.1	7200.0	6328.1	5512.5	4753.1	4050.0	3403.1	2812.5	2278.1	1800.0	1378.1	1012.5	703.1	450.0	253.1	112.5	28.1		18

Lines 15 & 16 correspond to 1 & 3 respectively. 17 – Moments from left end. 18 – Moments from right end.

Note: Loads are expressed in thousands of pounds and Moments in thousands of foot-pounds.

N.Y. C. & H.R.R.R.

Leased and Operated Lines.

New York, Jan 1, 1902. Office of Chief Engineer.

Moment Diagrams for Standard Loads.

Quantities for one rail.

Table II.

Live Load. The live load on each track shall consist of two consolidation engines coupled and followed or preceded by a train load, distributed as shown on the preceding diagram; or a special loading of 120,000 lb. equally distributed on a pair of driving axles spaced 7 ft., center to center, followed by a uniform train load of 4,500 lb. per lin. ft. of track, beginning 5 ft. from the axle, as shown.

The live load shall in all cases be placed in such position as to produce maximum strains in each member.

The load on each driver shall be considered as distributed by the rail over three cross-ties in proportioning the latter, and as distributed over a length of 4 ft. in proportioning solid floor bridges.

Painting. All painting shall be done with round brushes of the best quality obtainable in the market.

The standard red lead paint shall be mixed by the following formula for every five gallons of paint:

100 lb. pure red lead,
4 gal. pure raw linseed oil,
½ pint Japan, free from benzine.

A suitable quantity of the pigment shall be permitted for twenty-four hours to absorb its full capacity of raw linseed oil, thereupon it shall be worked or stirred to the consistency of a stiff paste. As much of this paste as may be needed for the next six hours or less shall then be thinned out with the requisite amount of oil required to give the ultimate proportions stated in the formula. In no case shall the paint mixed ready for use be more than six hours old at the time of application.

After erection, rivets driven in the field and all parts where the paint has been rubbed off in transportation or during erection, shall be well coated with red lead paint and the whole structure, except as hereinafter specified, shall thereupon be painted two heavy coats of N. Y. C. asphaltum varnish, which will be supplied to the contractor by the railroad company at the nearest station for eighty cents (80 c.) per gallon.

In all cases where the superstructures carry railroad tracks over highways or streets, unless otherwise specified by the chief engineer, the underside of said structures, exposed to the view of the public, including all trough floors, cross-girders, bracing, and interior columns, except the lower portion thereof

for a height of 5 ft. above the surface of the roadway or sidewalk, shall be painted, after erection, with two heavy coats of standard white paint, proportioned as follows, for every 5 gal. of paint:

42 lb. pure white lead in oil,
21 lb. pure white zinc in oil,
3 gal. pure raw linseed oil.

The lower portion of the interior columns, the bottom of the flanges of the side girders, and the columns supporting the side girders, and all other parts on the sides and top of said superstructures shall be painted with two heavy coats of N. Y. C. asphaltum varnish.

At least forty-eight hours must elapse between applying any two coats of paint.

Painting will not be allowed in wet or freezing weather.

All machine-finished surfaces shall be coated with a mixture of white lead and tallow as soon as finished.

The top of solid floor bridges of *I*-beams and plate and the top, bottom, and sides of trough floors shall not receive the two coats of asphaltum varnish after erection, but shall, instead, receive a special protective asphaltum covering to be furnished and applied by the railroad company.

Osborn Engineering Co. (No. 16).

Load on Curve. When the track is on a curve, both inner and outer girders are to be alike and proportioned for live load $W = [(M + B) \div 2B]P$, where M is center ordinate to curve, B is width center to center of girders, and P is live load at panel point considered.

The Excess Percentage of Weight allowed on wide plates of different thickness is as follows: For plates up to 75 in. wide, $\frac{1}{4}$ in. 10 per cent; $\frac{5}{16}$, 8; $\frac{3}{8}$, 7; $\frac{7}{16}$, 6; $\frac{1}{2}$, 5; $\frac{9}{16}$, $4\frac{1}{2}$; $\frac{5}{8}$, 4; over $\frac{5}{8}$, $3\frac{1}{2}$. For plates from 75 to 100 in. wide, $\frac{1}{4}$, 14; $\frac{5}{16}$, 12; $\frac{3}{8}$, 10; $\frac{7}{16}$, 8; $\frac{1}{2}$, 7; $\frac{9}{16}$, $6\frac{1}{2}$; $\frac{5}{8}$, 6; over $\frac{5}{8}$, 5. On plates over 100 in. wide, $\frac{1}{4}$, 18; $\frac{5}{16}$, 16; $\frac{3}{8}$, 13; $\frac{7}{16}$, 10; $\frac{1}{2}$, 9; $\frac{9}{16}$, $8\frac{1}{2}$; $\frac{5}{8}$, 8; over $\frac{5}{8}$, $6\frac{1}{2}$.

The specifications are accompanied by the rivet convention shown in Fig. 6, which since their construction suggests their meaning, are easily remembered, are simple, not easily confused on the drawing, and have been quite largely adopted by other designers.

Market Street Elevated Passenger Railway Company, Philadelphia (No. 18).

Tensile Tests for steel showing an ultimate strength within 5,000 lb. of that desired will be considered satisfactory, except that if the ultimate strength varies more than 4,000 lb. from

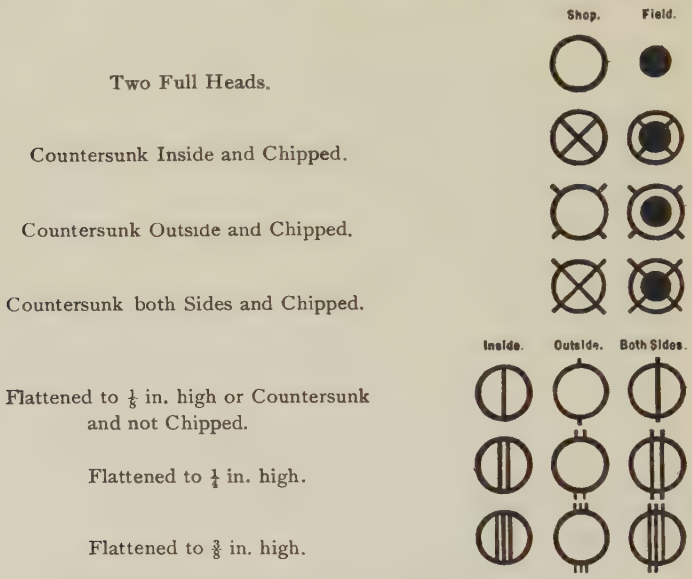


Fig. 6. Conventional Signs for Bridge Rivets.

that desired, a retest shall be made on the same gauge which, to be acceptable, shall be within 5,000 lb. of the desired ultimate.

Angles ¾ in. and less in thickness shall open flat, and angles ½ in. and less in thickness shall bend shut, cold, under blows of hammer without signs of fracture.

Cast Iron shall be tough, gray, and of uniform and close grain fracture, free from any white, mottled, or vitreous appearance. It shall be soft enough to be readily cut, drilled, and chipped, and when struck on a corner or edge with a hammer, it shall indent and not flake. Sample pieces 1 in. square, cast from the same heat and metal from which the castings are made, in sand moulds, shall be capable of sustaining on a clear span of 12 in. a central load of 2,400 lb. when tested in the rough bar.

Modifications in Elongation. The following modifications will be allowed in the requirements for elongation :

For each $\frac{1}{16}$ in. in thickness of the material below $\frac{5}{16}$ in. a deduction of $2\frac{1}{2}$ per cent will be allowed from the specified elongation.

For each $\frac{1}{8}$ in. in thickness above $\frac{3}{4}$ in. a deduction of 1 per cent will be allowed from the specified elongation.

For pins and rollers over 3 in. in diameter the elongation in 8 in. may be 5 per cent less than that specified above.

A Diminution in Cross Section or weight of any piece of steel of more than $2\frac{1}{2}$ per cent from that specified will be sufficient cause for rejection.

Plates will be accepted if they measure not more than .01 in. below the ordered thickness at the finished edges.

Payment for pound price contracts shall be by scale weights, and no allowance will be made in payment for excess weight exceeding $2\frac{1}{2}$ per cent of the weight computed from the plans, 1 cubic in. of rolled steel being assumed to weigh 0.2833 lb.

Field Connections. Holes for the connection of floorbeams to posts, and stringers to floorbeams, shall be sub-punched, and reamed with twist drills to a steel templet not less than 1 in. thick, or to other approved form of metal templet.

Web Splice plates and fillers under stiffeners shall be cut to fit within $\frac{1}{8}$ in. of flange angles.

Bed Plates. Where the ends of adjacent spans rest upon the same pier, the lower bed plate shall extend in one length under both bearings.

Two bed plates shall always be used where a pin bearing is not required, one to be riveted to the bottom flange of the girder, the other to be anchored to the masonry.

The Ends of the Top Flanges of half through girders shall be curved to meet the extreme end stiffeners when the girders are exposed to view.

Bracing. Lateral bracing shall be of such sections as will resist both tension and compression.

End sway bracing shall be placed at the ends of deck bridges, proportioned to transmit the accumulated upper lateral stresses to the substructure.

Intermediate sway bracing shall generally be placed on deck bridges at alternate panel points of the main lateral bracing. When the width between girders is large, so that the panels of lateral bracing are long, the sway bracing shall be placed at each panel point of the main lateral bracing.

Rivets. The minimum thickness of metal in which countersunk rivets are made shall be $\frac{3}{8}$ in.

The value of countersunk rivets shall be taken at one half the value of rivets with full heads.

Rivets must not be used in tension.

Smoke Protection. In cases of low headroom over steam railroads, an addition of $\frac{1}{16}$ in. of metal shall be made to the thickness of members exposed to the exhaust of locomotives.

In flanges with flange plates, this addition shall be made to the angles, and to the flange plate next to the angles.

Under plate girders over railroad tracks, wooden sheathing shall be placed to protect the structure from the exhaust of the locomotives, and shall consist of white pine not over 6 in. wide, dressed both sides to $\frac{7}{8}$ in. thickness, grooved and tongued. The boards may have a few small hard knots and sap not over $\frac{3}{4}$ of an in. in width on one side. The sheathing shall be divided into sections to facilitate removal, renewals, and access for painting where required, and shall be suspended by bolts and hangers or clamps, the battens and scantling to be of spruce of the best quality.

Painting. All recesses which would retain water or through which water could enter must be filled with thick paint, or some approved waterproof cementing compound before receiving the final painting. All surfaces so close together as to prevent the insertion of paint brushes must be painted thoroughly by using a piece of cloth instead of a brush.

The heads of all rivets driven in the field, and the heads and nuts of all bolts shall receive a coat of paint in advance of the two general coats after thorough cleaning.

All of the shop painting which may receive damage during transit or erection shall be properly cleaned and painted in advance of the application of the two general coats. Under no circumstances shall any of the painting be done in wet weather or over any wet surfaces, nor during freezing weather.

All work damaged during or after painting shall be repainted with the same precautions as are required in the specifications for the original work.

The wooden smoke protection sheathing shall receive three coats of approved metallic brown paint, on both top and bottom sides.

Southern Railway Company (No. 21).

Shoe Plates. The minimum thickness for shoe plates and bed plates respectively, is as follows: For spans under 65 feet $\frac{3}{4}$ in. and $\frac{7}{8}$ in., 65 to 100 ft. $\frac{7}{8}$ and 1 in., 100 to 125 ft. 1 and $1\frac{1}{4}$ in., above 125 ft. spans $1\frac{1}{8}$ and $1\frac{1}{2}$ in.

Countersunk Rivets. No value is allowed for $\frac{7}{8}$ in. rivets

countersunk in plates less than $\frac{5}{8}$ in. thick, nor to $\frac{3}{4}$ in. rivets countersunk in plates less than $\frac{1}{2}$ in. thick.

Braced Gussets with two angles at each edge shall be placed at all floorbeams and struts of through spans.

Lateral Connections must have at least four rivets each.

Rivets shall have a pitch not greater than sixteen times the thickness of the thinnest outside plate nor more than 6 in. nor less than three times the diameter. The distance from center of rivet to edge of plate must not be less than two diameters nor more than 4 in. or eight times the thickness of the plate. The diameter of the rivet in the leg of any angle must not exceed one fourth the length of the leg.

Sway Frames with at least four angles shall be placed at the ends of girders and 12 to 15 ft. apart, intermediately.

Flanges. Angles with a width of 6 in. or more shall be connected to the web plate with two rows of staggered rivets. Cover plates 13 in. or more in width, must have in each angle at least two rows of staggered rivets with a maximum pitch of $4\frac{1}{2}$ in. At the ends of cover plates, for a distance of at least twice the width of the plates, the pitch must not exceed four times the diameter of the rivet.

Union Pacific Railroad (No. 22).

Spacing of girders, center to center, shall be for spans of 60 ft. and under 7 ft., for 60 to 80 ft. spans 8 ft., for 80 to 100 ft. spans 9 ft.

Fixed Ends of spans shall be at the end connecting with trestle approach, at one abutment, at the lowest pier or at the down grade end.

Bed Plates under adjacent ends of consecutive spans shall be common to both. When the height of bed plate exceeds 6 in. it must be stiffened horizontally by separators or otherwise.

Bracing. Deck spans must have cross-frames or diaphragms at both ends and at intervals of 16 ft., intermediately.

Reaming. Rivet holes must be reamed in flanges and in the floorbeam connections.

Test Load. The completed bridge when tested by a train moving 60 miles an hour shall not deflect more than one fifteen hundredth of its span and must recover its original camber.

Pennsylvania Lines West of Pittsburg (No. 30).

Spacing of through girders with top flanges not more than 3 ft. above base of rail shall be 13 ft. c.c., deck girders $6\frac{1}{2}$ ft. c.c.

Excess Section must be provided when lateral force exceeds one third the sum of dead and live load stresses.

Lateral Struts must be proportioned for the direct stress and for the resultant of an initial stress of 10,000 lb. in the adjustable diagonal rods.

Minimum size of lateral rods is $1\frac{1}{4}$ in. square, of angles is 6 lb. per lin. ft.

Shop Assembling of all girder spans and reaming of all field connection rivet holes is required.

Maximum Variations of $2\frac{1}{2}$ per cent in the weight of steel plates and shapes are allowed except for plates more than 50 in. wide, which will be accepted when they do not overrun the estimated weight more than an additional $\frac{1}{2}$ per cent for each additional 10 in. in width, up to a total of 5 per cent.

Rivets must not have a grip more than five times the diameter. When directed, imperfect rivets shall be drilled out. Rivets over $\frac{7}{8}$ in. must not be hand driven. Web rivet pitch is determined by the kind of shear at the point under consideration, shearing, one kind, lb. per square in. $5{,}000\left(1+\frac{\text{min.}}{\text{max.}}\right)$, shearing, opposite kind, lb. per square in.

$$5{,}000\left(1-\frac{1}{2}\,\frac{\text{max. shear lesser kind}}{\text{max. shear greater kind}}\right).$$

The spacing of rivets in solid web girders shall be determined by the shear at the point under consideration, having regard to the kind of shear.

Shearing, one kind, $5{,}500\left(1+\frac{\text{min.}}{\text{max.}}\right)$ lb. per sq. in.

Shearing, opposite kind,

$$5{,}500\left(1+\frac{\text{max. shear of lesser kind}}{\text{max. shear of greater kind}}\right)\text{lb. per sq. in.}$$

The unit shear shall not exceed 75 per cent of the maximum allowed unit tension or compression stress in any member. The unit bearing stress shall not be more than twice the allowed unit shear. The unit bending stress shall not be more than $15{,}000\left(1+\frac{\text{min.}}{\text{max.}}\right)$.

Erie Railroad Co. (No. 31).

Shipped Complete. Deck plate girder bridges 40 ft. long and under shall be riveted together in the shops so as to have no field riveting.

End Bearing. In spans of 90 ft. and upwards great care must be taken that the web in the end panel bears on the bottom cover plate and sole plate, and a note to this effect must always appear on the shop plan.

Bed Plates. Bed plates must be designed to distribute the pressure with an extreme fiber strain in themselves not greater than 12,000 lbs. per sq. in. Cast iron bed plates shall be planed on both surfaces.

Spacing. Deck plate girders are spaced 5 ft. apart, center to center, for girders less than 5 ft. deep; when their depth is more than 5 ft. the spacing increases to a maximum of 7 ft.

Bending Stresses are assumed to be resisted entirely by the top and bottom flanges except for spans of 90 ft. or more, where $\frac{1}{8}$ the web is considered as effective for flange section. Shearing stresses are assumed to be resisted entirely by the web plate. In girders less than 50 ft. long the cross-section of the vertical legs of the flange angles is neglected in computing the flange section.

Center Girders in spans having more than one track are designed to carry $\frac{3}{4}$ of the total live load.

Web Splice Plates must each have a web-stiffening angle.

Michigan Central Railroad (No. 32).

Live Load Stresses in spans of less than 100 ft., when computed for the standard loading, must be multiplied by $[1\frac{1}{3} - (\text{loaded length} \div 300)]$.

Net Cross-Sectional areas are the maximum algebraic sums of such sections as are required to resist such live, dead, and wind load strains as may exist simultaneously, only as much of the wind load strain being considered as is in excess of $\frac{1}{3}$ the live load strain. When, however, other live load strains are combined with those due to friction between wheels and rails, the unit strains may be increased 15 per cent above those specified for live load, and for resisting these combined strains and those due to wind pressure, at the same time, dead load unit strains may be used, the whole of the wind pressure being considered in this case.

Sway Brace Frames in deck spans shall be spaced 40 ft. when there are bottom laterals, and 20 ft. when there are no bottom laterals.

Metal Floors supporting ballast shall be used, unless otherwise specified. The floor shall preferably consist of *I*-beams not more than 9 in. apart in the clear with $\frac{3}{8}$ in. plate riveted on top to support ballast and not counted in strength of beams. Each wheel load is assumed to be distributed over 3 lin. ft. of rail. All metal in contact with ballast must be covered with asphalt $\frac{1}{4}$ in. thick applied while the metal is heated to 125° Fah.

Splices. All web joints shall have a 13 in. splice plate on each side. The vertical legs of flange angles shall have reinforcement plates at joints of the web plates. Web plates shall be everywhere flush with backs of the flange angles, special attention being paid to this at bearings.

Variations in Weight of from $2\frac{1}{2}$ to 18 per cent are allowed in wide plates.

Lake Shore and Michigan Southern Railway (No. 33).

On Curves, deck girders shall be arranged to equalize the eccentricity of loading due to the curve. All girders shall be proportioned for the increase of load due to centrifugal force. In through spans, 1 in. additional clearance on each side shall be allowed for each degree of curvature, and $2\frac{1}{2}$ in. shall be allowed on the inside for each inch of super-elevation of track.

Intermediate Transverse Frames are not more than 20 ft. apart.

Splices. In spans of 70 ft. or less the flanges shall not be spliced. Flange angles shall not be spliced adjacent to cover plate splices; their splices shall be made with angles of equal sectional area and only one angle in a flange shall be spliced at the same point. Each web splice shall have one set of stiffener angles.

Lateral Angles shall not be less than $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ in. and shall have at least four rivets at each end.

Shipping. Deck spans of less than 75 ft. shall be riveted up complete at shop.

Floors shall preferably be of steel beams and stringers carrying wooden cross-ties and rails. Extra thin floors may be of solid or trough construction, with or without floorbeams, ballast, or concrete filling. Solid floors shall be preferably made with rolled *I*-beams and plate not less than $\frac{3}{8}$ in. thick for floor.

New York, New Haven, and Hartford Railroad (No. 34).

On Curves the bridge axis shall bisect the middle ordinate and be parallel to the chord of the curve.

Girders Spaced 12 ft. apart on centers in half through spans must not project more than 2 ft. vertically above the base of rail, and the width of top flanges must not exceed 15 in.

Rivets carrying calculated stress and whose grip exceeds four diameters shall be increased in number 1 per cent for each additional $\frac{1}{16}$ in. in grip.

Imperfect Masonry Bearings shall be corrected with rust cement or with Portland cement mortar.

Long Island Railroad (No. 37).

Bracing. Deck spans shall have cross-frames at both ends and intermediately from 12 to 15 ft. apart. Bottom laterals shall be used only on curves sharper than 3 degrees.

New York, Chicago, and St. Louis Railroad Company (No. 24).

Viaducts. Viaducts shall be composed of deck plate girders spaced 8 ft. center to center, and having alternate spans of 30 ft. and 60 ft. respectively. All girders shall have the same depth, and they shall rest directly upon the caps of the columns forming the towers. The towers shall be composed of Z-bar columns thoroughly braced with diagonals and struts. Each member of the bracing shall be made of two or more angles laced together and riveted to the columns by means of gusset plates.

Tower Bracing. In trestle towers the transverse bracing and posts shall be proportioned to resist lateral pressure in addition to the stresses from live and dead loads as follows:

a. At the post cap, a force of 1,000 lb., with 30 lb. per lineal foot of structure in addition for each foot in depth of floor and girder measured from top of rail to bottom of girder of the 60-ft. span.

b. A force of 300 lb. per lineal foot of structure applied 8.5 ft. above the base of rail.

c. A force of 2,700 lb. applied at each transverse strut, if any.

Unit Strains. All parts of the structure, not otherwise specified, shall be proportioned according to the following unit strains, modified, in the case of direct strains, by the coefficient

$$1 + \frac{\text{min. strain}}{\text{max strain}},$$

and for alternate strains, by the coefficient

$$1 - \tfrac{1}{2}\left(\frac{\text{max. strain lesser kind}}{\text{max. strain greater kind}}\right):$$

Tension	7,500 lb. per	sq. in.
Shear	6,000 "	"
Bearing	12,000 "	"
Bending on Pins	12,000 "	"

Bracing. All bracing not subject to static live-load strains shall be proportioned according to the following unit strains:

Tension	15,000 lb. per	sq. in.
Compression	$\dfrac{15{,}000}{1 + \dfrac{l^2}{36{,}000\, r^2}}$	
Shear	12,000 "	"
Bearing	24,000 "	"

Rivets in single shear, $6000\left(1 + \frac{\text{min.}}{\text{max.}}\right)$, in bearing $12{,}000\left(1 + \frac{\text{min.}}{\text{max.}}\right)$. The number of rivets determined by this formula is to be increased 25% for all field driven rivets.

Bracing. All deck girders shall have top and bottom lateral bracing extending from end to end of the span, no brace making an angle of less than 40 degrees with the girders. All deck spans shall be braced at the ends and at intermediate points not more than 15 ft. apart with rectangular cross-frames, each

DIAGRAM OF 136 TON ENGINE.

Class U—Waddell.

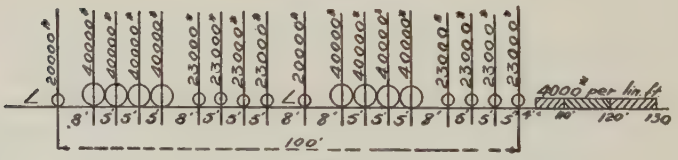


Fig. 7.

made of angles and plates, with two struts and two diagonals. No angle smaller than $3 \times 3 \times \frac{3}{8}$ in. shall be used in the diagonals or cross-frames. Specification No. 24.

Housing for Roller Nests. Roller nests for expansion bearings shall be protected by metal housings bolted to the shoe

plates, so as to exclude all dirt or rubbish from contact with the rollers.

Masonry Plates. Where two spans rest upon the same masonry a continuous plate not less than $\frac{5}{8}$ in. thick shall extend under the two adjacent bearings.

Floorbeams and Stringers. The ends of all floorbeams and stringers shall be faced true and square and to correct lengths. Allowance shall be made in the connecting angles for such facing without reducing the required section.

Floor stringers shall be spaced not less than 6 ft. 6 in. center to center.

Tension Members. Angles subject to direct tension shall be connected by both legs to the supporting members.

Web Splices. The webs of plate girders shall, when spliced, have a universal plate on each side of the web, of the same thickness as the web, and of sufficient width to take at least two rows of rivets on each side of the joint. Enough rivets shall be used to transmit the total shear. Web splices shall not be used at the center of the girder.

Web plates shall be arranged so as not to project beyond the faces of the flange angles, and so as not to be more than $\frac{1}{16}$ in. below the face of these angles at any point on top.

Pennsylvania Railroad Company (No. 23).

Tests. Separate tests shall be made from sheared plates, universal mill plates, beams, angles, channels, Z-bars, flats, rounds, pin steel, and eyebar steel. The six requirements are for bending, drifting, ultimate strength, elastic limit, elongation, and reduction of area. If samples fulfill five of the six requirements, duplicate tests may be made, and if it and the average of both tests are successful, the melt may be accepted.

Stresses and Sections. The equivalent static stress in any member is $M\ (1 + k)$ where M is the maximum calculated tension or compression, and k is determined from the formula for stresses subject to reversal,

$$k = \left(\frac{2 - 3R - R^2}{2}\right).$$

For stresses not subject to reversal $k = 1 - 2R + R^2$.

$$R = \frac{m}{M},$$

and m is the minimum calculated stress in members subjected to tension only, or to compression only, or is the maximum calculated stress of the lesser kind in members subjected to reversal of stress. Values of k are tabulated to correspond with values of R from 0.00 to 1.00, and range from 1,000 to 2,000.

Static Stress. All members are proportioned so that the static stress, $M(1 + k)$, shall not cause the tensile unit stress to exceed 16,000 lb., nor the compressive unit stress to exceed 16,000 lb. diminished by the usual radius of gyration formula. In stresses including wind pressure or momentum the increment k shall be omitted.

Centrifugal Force. The stress due to centrifugal force shall be regarded as live load, and when necessary additions shall be made to the sections of girder flanges until the unit stress does not exceed 18,000 lb. in tension or 18,000 lb. properly reduced in compression. In lateral bracing the stress due to centrifugal force shall be increased 50 per cent and the increment k shall be omitted. In deck spans where the curvature exceeds 6 degrees, the lower lateral bracing shall be designed to carry half the stress due to centrifugal force, and sway bracing shall be introduced to transfer the stress from the upper to the lower system.

Viaduct Stresses. In the posts of trestle towers, when the maximum stresses due to wind and train momentum, plus the maximum stresses from vertical loading and centrifugal force, properly increased, exceed 20,000 lb. per sq. in., reduced by the compression formula, additions must be made to the sections until this limit is not exceeded.

Future Increase of Loads. To insure the stability of bridges under increased live loads, a live load shall be assumed 100 per cent greater than that previously provided for in this specification. If the resultant stress, $M(1 + k)$, produces a stress per square inch in any member more than twice the permissible unit stress previously specified, additions must be made to the sections until that limit is not exceeded.

Rivets. Rivets with countersunk heads shall be assumed to have three fourths the value of corresponding rivets with full heads. Where the floor is carried on the top flanges of deck girders, the web rivets in the top flange angles must be proportioned for a 15-ton driver load distributed over three ties.

Sway Bracing. Deck spans shall have, at each panel, diagonal sway bracing of sufficient strength to carry half the maximum stress increment due to wind and centrifugal force.

General Specifications for Steel Railroad Bridges, Bogue and Buel (No. 25).

When limited head room necessitates a less depth than $\frac{1}{12}$ of the span, the girder shall be so proportioned that the moment of inertia will be the same as would be required for a girder of the same span with a depth of $\frac{1}{12}$ of the span.

Viaducts and Trestles. Each trestle tower shall in general consist of two adjoining bents, each of which latter will generally consist of two supporting columns. The columns will have a batter transversely not less than 6 vertical to 1 horizontal for single track, and not less than 8 vertical to 1 horizontal for double track. All towers shall be well braced transversely to resist wind pressure, and, if on a curve, centrifugal force; and shall be braced longitudinally to resist the maximum possible stresses due to traction or setting brakes. They shall also have diagonal lateral bracing in horizontal planes in each story and at the feet, and shall have struts between the feet of the columns both in the planes of the bents and of the sides of the towers. The columns shall be anchored to the pedestals or masonry with anchor bolts sufficient to resist double the calculated stresses that may come upon them. The tower spans of high trestles shall not be less than 30 ft. center to center of bents, and shall alternate with free spans between the towers which may be of economic length. Both the tower spans and free spans will generally consist of plate girders.

Connections. The tower girders shall be riveted between transverse girders which in turn shall be riveted between the columns. The free girders shall be riveted to the transverse girders at one end and shall be provided with expansion joints at the other end.

A live load, consisting of two consolidation engines followed by a uniform train load, which, unless otherwise specified, shall be according to the weights and spacing shown on the diagram entitled "Typical Loading" attached hereto; or a concentrated loading of 120,000 lb. on two axles spaced 6 ft. center to center. This is the E-50 loading of the Cooper Series.

If heavier or lighter loadings are desired, they will be specified in the letter of invitation and will be designated by the index numbers, as E-40, E-45, E-55, E-60, etc., and they shall be proportional to the E-50 loading, specified above and shown on diagram, with the same wheel base.

Unless otherwise directed by the chief engineer, the following uniform live loads per lineal foot of track may be substituted for the typical train loadings above specified:

A live load, per lineal foot of track, of "T" divided by (the cube root of the length of the span in feet plus the sixth root of the length of the span in feet). The values of "T," approximately corresponding to the loadings of the "E" Series, are as follows:

T = 40,000 for E-40 loading.
T = 45,000 " E-45 "
T = 50,000 " E-50 "
T = 55,000 " E-55 "
T = 60,000 " E-60 "

The table hereto attached gives the load per foot of track for "T" = 50,000 for spans of 10 ft. to 500 ft. For "T" equal to any other value, the load per foot of track will be proportional.

The value of "T," or the index of the loading by this series, is approximately equal to the weight on each driving axle in pounds, or ten (10) times the following load per lineal foot of track.

These loads shall be so placed as to produce the greatest stress in each member. Wherever the live and dead load stresses are of opposite character, only 70 per cent of the dead load stress shall be considered as effective in counteracting the live load stress.

An allowance for impact or dynamic effect of live loads, when there is no reversal of stress, equal to the live load multiplied by the maximum stress, divided by the maximum stress plus the minimum stress in the member considered. When a member is subject to reversal of stress, this allowance (to be applied to the live load stress of each kind) shall be equal to that live load multiplied by 1 plus twice (the maximum stress of lesser kind divided by the maximum stress of greater kind). The stresses due to dead and live load only, shall be used in calculating the above allowances.

Stresses due to momentum, or tractive force of engines and stresses due to centrifugal force, will be increased by an amount to provide for impact or dynamic effect one-half the amount provided for above, and shall be treated as distinct from the impact added to live load stress.

TABLE 27. — UNIFORM LIVE LOADS IN POUNDS FOR $T = 50{,}000$. Other Loadings Proportional.

Span in Feet. S	$\sqrt[3]{S}$	$\sqrt[6]{S}$	$\sqrt[3]{S} + \sqrt[6]{S}$	Uniform Live Load per Foot of Track. $\frac{50{,}000}{\sqrt[3]{S} + \sqrt[6]{S}}$
10	2.1544	1.4678	3.6222	13804
15	2.4662	1.5704	4.0366	12387
20	2.7144	1.6475	4.3619	11463
25	2.9240	1.7100	4.6340	10790
30	3.1072	1.7627	4.8699	10267
35	3.2711	1.8086	5.0797	9843
40	3.4200	1.8493	5.2693	9489
45	3.5569	1.8860	5.4429	9186
50	3.6840	1.9194	5.6034	8923
55	3.8030	1.9501	5.7531	8691
60	3.9149	1.9786	5.8935	8484
65	4.0207	2.0052	6.0259	8298
70	4.1213	2.0301	6.1514	8128
75	4.2172	2.0536	6.2710	7973
80	4.3089	2.0758	6.3847	7831
85	4.3968	2.0969	6.4937	7700
90	4.4814	2.1169	6.5983	7578
95	4.5629	2.1361	6.6990	7464
100	4.6416	2.1544	6.7960	7357
110	4.7914	2.1889	6.9803	7163
120	4.9324	2.2209	7.1533	6990
130	5.0658	2.2507	7.3165	6834
140	5.1925	2.2787	7.4712	6692
150	5.3133	2.3051	7.6184	6563
160	5.4288	2.3300	7.7688	6436
170	5.5397	2.3536	7.8933	6334
180	5.6462	2.3762	8.0224	6233
190	5.7489	2.3977	8.1466	6138
200	5.8480	2.4183	8.2663	6049
225	6.0822	2.4662	8.5484	5849
250	6.2996	2.5099	8.8095	5676
275	6.5030	2.5501	9.0531	5523
300	6.6943	2.5873	9.2816	5387
325	6.8753	2.6221	9.4974	5265
350	7.0473	2.6547	9.7020	5154
400	7.3700	2.7150	10.0850	4960
500	7.9400	2.8170	10.7570	4650

TABLE 28. — DECK PLATE GIRDERS, SINGLE TRACK.

Span in Feet.	Weight in Pounds.	
	Per Foot.	Per Span.
Out to Out $= L$.	$14.2\,L + 140$.	$14.2\,L^2 + 140\,L$.
35	637.5	22,300
50	849.6	42,480
65	1,063.0	69,100
80	1,275.0	102,000
95	1,489.0	141,450

TABLE 29.—COEFFICIENTS OF IMPACT WHEN THERE IS NO REVERSAL OF STRESS.

$\frac{\text{min.}}{\text{max.}}$	Per cent of Live Load.	$\frac{\text{min.}}{\text{max.}}$	Per cent of Live Load	$\frac{\text{min.}}{\text{max.}}$	Per cent of Live Load.	$\frac{\text{min.}}{\text{max.}}$	Per cent of Live Load.
.00	1.0000						
.01	.9901	.26	.7937	.51	.6623	.76	.5682
.02	.9804	.27	.7874	.52	.6579	.77	.5650
.03	.9709	.28	.7813	.53	.6536	.78	.5618
.04	.9615	.29	.7752	.54	.6494	.79	.5587
.05	.9524	.30	.7692	.55	.6452	.80	.5556
.06	.9434	.31	.7634	.56	.6410	.81	.5525
.07	.9346	.32	.7576	.57	.6369	.82	.5495
.08	.9259	.33	.7519	.58	.6329	.83	.5464
.09	.9174	.34	.7463	.59	.6289	.84	.5435
.10	.9091	.35	.7407	.60	.6250	.85	.5405
.11	.9009	.36	.7353	.61	.6211	.86	.5376
.12	.8929	.37	.7299	.62	.6173	.87	.5348
.13	.8850	.38	.7246	.63	.6135	.88	.5319
.14	.8772	.39	.7194	.64	.6098	.89	.5291
.15	.8696	.40	.7143	.65	.6061	.90	.5263
.16	.8621	.41	.7092	.66	.6024	.91	.5236
.17	.8547	.42	.7042	.67	.5988	.92	.5208
.18	.8475	.43	.6993	.68	.5952	.93	.5181
.19	.8403	.44	.6944	.69	.5917	.94	.5155
.20	.8333	.45	.6897	.70	.5882	.95	.5128
.21	.8264	.46	.6849	.71	.5848	.96	.5102
.22	.8197	.47	.6803	.72	.5814	.97	.5076
.23	.8130	.48	.6757	.73	.5780	.98	.5051
.24	.8065	.49	.6711	.74	.5747	.99	.5025
.25	.8000	.50	.6667	.75	.5714	1.00	.5000

$$I = \frac{M}{M + m}$$

I = Impact = per cent of live load.

M = Maximum stress.

m = Minimum stress.

For members in which the maximum stresses are produced by loads on two or more tracks, the above allowances shall be reduced by twenty (20%) per cent.

An allowance for wind loads and lateral vibration, as follows:

Dead Load, 150 lb. per lin. ft. on the unloaded chord, and 200 lb. per lin. ft. on the loaded chord.

Moving Load, 400 lb. per lin. ft. on the loaded chord, applied at a distance of six feet above the base of rail.

In calculating the righting moment for wind reactions, the weight of the train will be assumed at 800 lb. per lin. ft. In trestle towers the bracing and columns shall be proportioned to resist the above specified wind loads and in addition a horizontal load of 180 lb. for each vertical foot of bent. In calculating wind stresses in trestle bents carrying double track, the conditions with or without loads on either one or both

tracks, and with the maximum train load and with a train load of 800 lb. per lin. ft., shall be considered, and the maximum stresses produced by either condition shall be used.

Centrifugal Force. When the structure is located on a curve, a centrifugal force acting 5 ft. above the base of rail and treated as live load shall be computed by the following formula for a maximum train on each track moving at a speed of $60 - 3D$ miles per hour, D being the degree of curvature. $C = \frac{wv^2}{32.2\,r}$, in which C = centrifugal force, w = weight, v = velocity in feet per second, and r = radius of curve.

Shearing. On webs of girders and beams, $16{,}000 - 100\frac{d}{t}$ lb. per square inch in which d = the clear distance between flange angles or stiffener angles, and t = the thickness of the web, both in inches.

Western Pacific Railway Company (No. 28).

Depth of Girders. Plate girders shall generally have a depth of from $\frac{1}{10}$ to $\frac{1}{12}$ of the span. When limited head room necessitates a less depth than $\frac{1}{12}$ of the span, the girder shall be so proportioned that the moment of inertia will be the same as would be required for a girder of the same span with a depth of $\frac{1}{12}$ of the span.

Sub-Punching and Reaming. In structural steel, wherever used, except in lateral, longitudinal, and sway bracing, all holes shall be sub-punched and reamed with twist drills, unless otherwise marked on plans.

In sub-punched and reamed work the holes shall first be punched $\frac{1}{8}$ in. smaller and then reamed $\frac{1}{16}$ in. larger than the normal size of the rivets. The sharp edges must be removed from all reamed or drilled holes.

Shearing. On webs of girders and beams, $16{,}000 - 100\frac{d}{t}$ pounds per square inch, in which d = the clear distance between flange angles or stiffener angles, and t = the thickness of the web, both in inches.

Stiffeners. All web plates shall be stiffened at the inner edges of end bearing, and at all points of local concentrated loadings. Intermediate stiffeners will be used where required, so that the shearing stress per square inch shall not exceed that given by the formula of the preceding paragraph.

Flanges and Stiffeners. In girders with flange plates, the angles shall form as large a proportion of the flange area as practicable. When flange plates are not of the same thickness, they shall diminish in thickness outward from the angles. The first flange plate of the top flange shall extend the full length of the girder, and all flange plates must be at least one foot longer than their theoretical length. The webs of all plate girders shall have stiffeners over bearing plates and at all points of local concentrated loadings. The projecting leg of the end stiffeners shall be just one size less than the horizontal leg of the flange angles and there shall be two angles over each end of each bearing.

Splices. Web plates shall be spliced by a plate on each side with sufficient rivets and section to transmit the shear, and in addition shall be spliced by side plates near the flange angles with sufficient rivets and section to make up the loss in the moment of inertia of the girder section due to the web splice. When necessary to splice the flange angles they shall break joints and be spliced by angle splices in pairs on both sides of the flange.

Linseed Oil. Linseed oil shall be pure, aged six months and double kettle-boiled. It shall be free from all adulterations, and not contain any fish oil nor mineral oil, and no drier, except turpentine. No more than 10 per cent of turpentine will be allowed either in linseed oil or in paint.

Iron Oxide. The iron oxide shall contain not less than 90 per cent of sesquioxide of iron and be practically free from sulphur and moisture. It shall be finely ground, and, if lumpy, shall be roasted, to drive off the water.

Paint. Paint will consist of pure, double kettle-boiled linseed oil, iron oxide and pure turpentine, mixed in the proportions of 8 pounds of red oxide of iron to a gallon of oil, and not over one gallon of turpentine to each ten or twelve gallons of oil. It shall be well mixed.

Denver and Rio Grande Railroad System.

Our manner of handling this work is to give our consulting engineer information relative to size, etc. He then prepares stress diagrams and general drawings showing make-up of the members, and these, together with the specifications of the Pennsylvania Lines West of Pittsburg of April, 1897, are sub-

mitted to the manufacturers for tenders, which are usually based on pound price. The manufacturer to whom the contract is awarded then makes up his own shop drawings, and we secure the services of an inspector to follow the metal through the shop and see that all is properly loaded on cars, and the work is paid for on the basis of the shipping weight. After the drawings have been completed by the manufacturer the tracings, or originals as we call them, become the property of the railroad company, the manufacturers retaining as many copies as they care to for their files and record.

W. A. Morey,
Engineer of Bridges and Buildings.

July 26, 1904.

PART II

ILLUSTRATIONS AND DESCRIPTIONS OF STANDARD DESIGNS AND DETAILS

PART II

ILLUSTRATIONS AND DESCRIPTIONS OF STANDARD DESIGNS AND DETAILS

A NUMBER of important railroads have standardized the detailing of plate girder spans and have established designs as uniform and rational as their specifications, from which indeed, they are frequently inseparable. Some railroad bridge engineers have gone so far as to design a complete series of deck and through spans covering all lengths adapted to plate girder construction, and computing stresses and dimensioning the structures for span increments of 5 or 10 ft. so they will suffice for any opening, selecting the design usually for the length next greater than the case in point if the latter is an odd distance.

For other railroads general details and connections have been worked out covering all features of construction and are issued as standards, and the given structure has the essential data collated and notes and diagrams presented referring the draughtsman or builder to the separate details which are simply assembled to make the required span, thus allowing rather more liberty and variation in the combination of details or their modification to suit varying conditions.

In some cases, general types have been adopted, and all structures of the same class conform to them in all essentials, although complete new drawings are required in every case. These standards, even if not specifically recognized as imperative, are closely followed.

The most radical improvement in the detailing of plate girders has been in the rational treatment of end bearings to promote stability, to provide for temperature displacements, to secure definite bearings, to concentrate the loads, to avoid excessive stresses in the steel or the masonry, to allow deflections to occur without injury to the structure or the development of serious secondary stresses, to protect the bearings and to provide for their maintenance in good condition.

Efficient web stiffeners are furnished to take shear over end

bearings, and the lateral stability of the girder is increased when necessary by extending the bearings transversely and bracing them to the girder webs with vertical diaphragms. The loads are concentrated on shoes which raise the lower flanges above the substructure and facilitate their maintenance and protection and provide for suitable masonry supports.

For short spans and light loads, where deflection is negligible, the fixed end shoes may be seated directly on the masonry, but for long spans or heavy loads provision must be made for longitudinal temperature displacement at one end and for preserving uniformly distributed pressure on the masonry when the girder is distorted by deflection. Longitudinal displacement is provided for by sliding or rolling ends, both being fixed against transverse displacement so as to afford anchorage for transverse lateral stresses. Deflections are met with pin bearings or their equivalents which require both shoes and pedestals. Considerable skill and ingenuity have been displayed in making a variety of riveted and cast shoes and pedestals, the principal examples of which are included in the accompanying illustrations, and a much larger number is shown in Chapters XIII. to XXIII. inclusive of Vol. II. of "Types and Details of Bridge Construction."

CHAPTER VI

END BEARINGS.

Chicago, Milwaukee and St. Paul Railway Standards.

SPANS of less than 70 ft. have sole plates riveted to the bottom flanges and seated on cast-iron webbed pedestals. The tops of the pedestals and the bottoms of the sole plates are planed, and they are connected by two $1\frac{1}{2}$-in. bolts passing through slotted holes in the sole plates at both ends of all girders so that the expansion and contraction movements may take place at both ends. The abutment pedestals for through girders are made in three sizes, proportioned to raise the girder a convenient height above the substructure, thus saving masonry, affording an opportunity to keep the seats clear, and avoiding the accumulation of rubbish and moisture around the lower flanges, while reducing the maximum masonry pressure to 250 lb. per square inch. The smallest size for through girders weighs 425 lb. and is used for girders 30, 35, and 40 ft. long. The largest size weighs 655 lb. and is used for 55, 60, and 65-ft. girders. The intermediate size has a 23×27-in. base, weighs 570 lb., is used for 45 and 50-ft. girders, and is similar to the large size except in the dimensions of the top and bottom plates.

The pedestals for the corresponding lengths of deck girders are like those illustrated, except that the anchor bolt holes are in the outer corners of the base plates, and their dimensions and weights vary. The pedestal for the 25, 30, 35, 40, and 45-ft. girders weighs 395 lb., that for the 50 and 55-ft. girders weighs 535 lb., and that for the 60 and 65-ft. girders weighs 595 lb.

All pier pedestals receive the ends of the two adjacent girders, and are similar to the casting illustrated, which is used for 30, 35, and 40-ft. through girders and weighs 845 lb.

All pedestals have $1\frac{3}{4}$-in. cored holes, and have the lower surface planed if it is not cast true within $\frac{1}{16}$ in.

Type E girders have shallow floors with the ties supported on shelf angles riveted to the webs. They are seated on bed plates on steel sills made of pairs of channels, back to back,

with transverse channel diaphragms connecting their webs at centers of bearings. Other shallow floor girders are seated on cast-iron pedestals similar to the one shown for 8 to 12-ft. spans, which weighs 220 lb.

For spans of 70 to 90 ft., triple-web cast-steel shoes are used, which have their webs connected by a pair of transverse diaphragms. The shoes weigh 195 lb. each and engage corresponding 285-lb. pedestals with 4-in. pins. The center webs in shoes and pedestals are in the same plane and have half-hole bearings the side webs clear each other and have full-hole bearings. In most existing spans at the fixed end the pedestal is seated on a bolster, and at the expansion end on a set of four 8-in. segmental rollers connected to the top and bottom end strips, with gudgeons turned to driving fit. Those in the end rollers have their extremities threaded to receive nuts, securing them to the strips; those in the intermediate rollers have shouldered ends flush with the strips. The rollers are seated on a bed made with rails riveted to a $1\frac{3}{8}$-in. plate. The lower flanges of the rails have their edges beveled, and one of them is cut away nearly to the web. They are thus fitted exactly, and $\frac{5}{8}$-in. rivets are driven through the wide side of the flange only. A $\frac{15}{16} \times \frac{3}{8}$-in. guide rib is planed in the head of the center rail, and the heads of the other rails are planed down $\frac{5}{16}$ in. to a uniform height. The center rail is an 85-lb. section $5\frac{3}{16}$ in. high, and the side rails are 75 lb. $4\frac{13}{16}$ in. high.

The pier bed is like the abutment bed shown, except that it is 3 ft. 9 in. long. The end rollers are secured to their side strips by $1\frac{1}{4}$-inch stud bolts having nuts, washers, and collars; the intermediate ones are secured by stud bolts shouldered at the outer ends to engage reduced holes in the side strips beyond which they do not project.

The old roller bearings for girders 70 to 90 ft. long have been recently replaced by new designs, having 4-in. pin bearings in triple-web cast-steel shoes and pedestals. The center webs of shoe and pedestal are in the same plane and have half-holes for the pin; the outside webs clear each other and have full holes locking the shoe and pedestal together. The webs of the shoe and pedestal are connected by transverse vertical diaphragms which very nearly inclose the space surrounding the pin, and exclude most of the dust and moisture. All shoes are alike at both fixed and expansion ends; the corresponding pedestals are cast from the same pattern, but those for the ex-

pansion ends have thicker base plates, recessed and finished to receive the phosphor-bronze disk. The upper surfaces of all shoes are planed for bearing on the girder flanges, and the bolt holes are drilled or reamed. One shoe and one pedestal together weigh about 520 lb. exclusive of the 930-lb. cast-iron bed plate.

In a recent letter, referring to these bearings, Mr. C. F. Loweth, engineer and superintendent of bridges and buildings of the railway, says :

"You will note that the bearing comprises two pin-connected cast-steel shoes, the upper one bolted to the girder. These provide for the deflection of the girder. They are supported on large iron castings resting on the masonry, with sheet lead interposed. To provide for expansion there is placed between the lower cast-steel shoe and the cast-iron bed plate a phosphor-bronze plate doweled into the bed plate. At the fixed end of spans the bronze plate is omitted. The coefficient of friction allowance on the bronze plate is 25 per cent.

"Anyone who has had to do with the maintenance of bridges must have had forcibly called to his attention the inefficiency of rollers, especially the old type of small rollers. Not infrequently the rollers and roller beds are so rusted as to prevent any rolling motion whatever, and the surfaces so grooved with rust as to prevent sliding motion. In taking down old spans it is not infrequently found that the rollers are so rusted as to be appreciably flattened, and deep grooves are rusted in roller beds and roller shoes where in contact with the rollers. It seems practically impossible to maintain such expansion joints in working condition. Paint can with the utmost difficulty be gotten on to the surfaces, and runs into the angles formed by the several surfaces of the rollers and roller beds and hardens, acting as wedges. Segmental rollers of large diameter are much better, but at the best are far from being ideal, and, except for their fewer number and the larger spaces about them for inspection and cleaning, are just as difficult to maintain in working condition as small, round rollers. I think one would not be far astray in venturing the statement that 95 per cent, or even more, of all roller bearings are practically neglected.

"I expect the bronze sliding joint to require no more care than the rest of the structure, and to be efficient as an expansion joint at the start and throughout the lifetime of the structure.

"You will note that I have placed the expansion joint well

above the surface of the masonry and where it will be less exposed to dust, though this is less important with this kind of expansion detail than where rollers are used. 'The Engineering Record' and other papers frequently contain illustrations of roller bearings in which the rollers are scarcely raised above the masonry. It is out of the question to keep them clean under such conditions. It would be much better if they were raised well above the masonry. The substitution of deep castings between the rollers and the bridge seat frequently results in economy, the castings costing less than the masonry saved, and in addition there is a better distribution of the load over rollers and on the masonry."

Northern Pacific Railway Standards.

Standard single-track plate-girder bridges of 50 to 65-ft. span on the Northern Pacific Railway have special hinge bearings. Cast-steel shoes are bolted to the under sides of the lower flanges and engage the convex surfaces of cast-steel pedestals, to which they are connected by vertical bolts countersunk in the pedestal bases with clearance to allow a slight longitudinal rocking motion. This provides for the deflection of the girder and preserves a full concentric bearing on the cylindrical surface when the girder is in a slightly inclined position. The bearings are hollowed in the center so as to give contact only over the two webs of the pedestal. The pedestal is loose on the bed plate, which is anchor-bolted to the masonry, and has narrow horizontal side plates fillered up to project over the pedestal flanges and lock them transversely and vertically in place. For spans up to 60 ft. riveted steel-plate shoes and pedestals are also designed as alternative standards for use on timber piers and abutments.

Plate-girder spans of 65 to 100 ft. have double web cast-steel shoes with horizontal top plates bolted to the lower flanges and 5-in. transverse pins which engage the corresponding outside web plates of cast-steel pedestals like the shoes. The roller end pedestals are seated on nests of segmental rollers with top and bottom side plates, which engage the edges of the pedestals and base plates and have an interlocking center spur and notch which limit the angle through which the rollers can rock. The $\frac{7}{16}$-in. bed plate has a $\frac{1}{4}$-in. sheet lead packing between it and the masonry to which it is anchor-bolted, and has

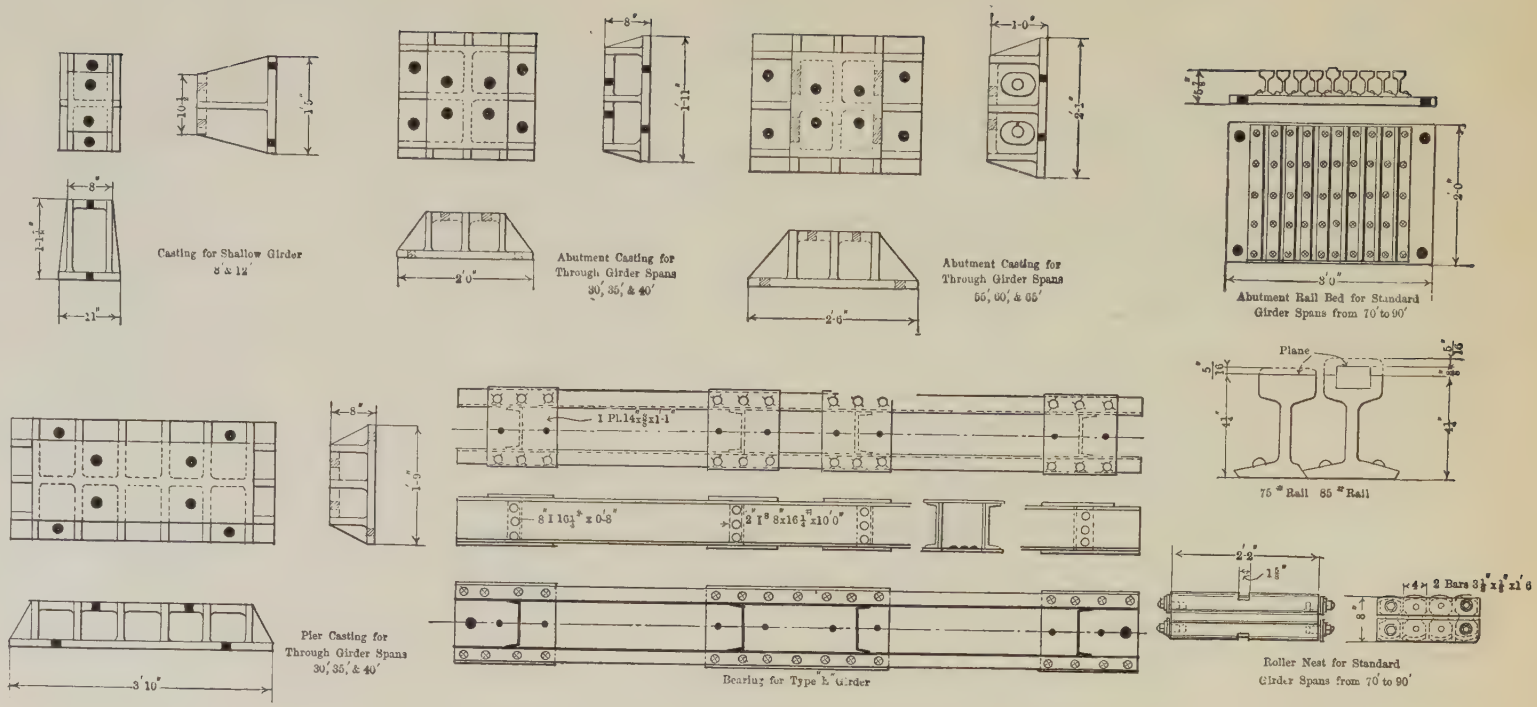

Casting for Shallow Girder
8′ & 12′
Abutment Casting for
Through Girder Spans
30′, 35′, & 40′
Abutment Casting for
Through Girder Spans
55′, 60′, & 65′
Abutment Rail Bed for Standard
Girder Spans from 70′ to 90′
Plane
75 ℔ Rail 85 ℔ Rail
1 Pl. 14″x⅜″x1′-1″
8 I 16¼″ x 0′-8″
2 I 8″x16½″x10′0″
Pier Casting for
Through Girder Spans
30′, 35′, & 40′
Bearing for Type h Girder
Roller Nest for Standard
Girder Spans from 70′ to 90′

Fig. 10.

riveted to the upper side a set of longitudinal T-rails with one side of each lower flange cut off so their heads are nearly in contact, and planed down to a continuous roller bearing.

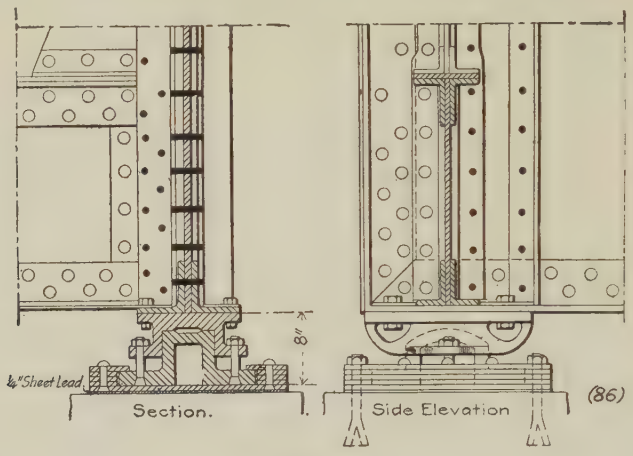


Fig. 11.

The fixed and expansion end shoes and pedestals are alike, and the former are seated on cast-iron bolsters with longitudinal and transverse webs which have a height equal to the combined

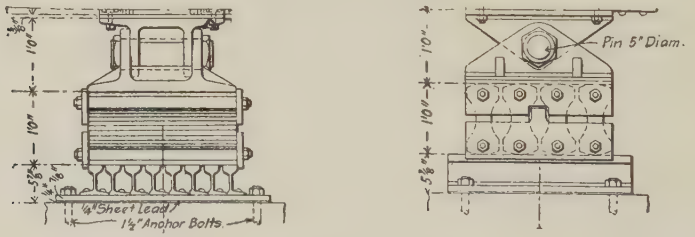


Fig. 12.

heights of the rollers and roller beds. The pins have full-hole bearings in the shoes and pedestals, and the latter are bolted to the bolsters, thus securing the girders against displacement in any direction.

In former standards the expansion ends of plate girders 70 ft. and more in length were seated on rocker bearings made with a riveted shoe and pedestal connected with curious double-cylindrical pins. The shoes had triple webs, with their ends covered by bent transverse plates, to exclude dirt, and were bored to give about 220 degrees bearing on the pins. The

pedestals were like the shoes reversed and made with wider base plates, so as to allow for the outside anchor bolts. They were seated on lead bearing plates, and the web cover plate on the outside end was bolted on instead of being riveted, in order to allow its removal to facilitate inspection and cleaning. There was a clearance of 1 inch between the shoe and pedestal which provided amply for longitudinal displacement.

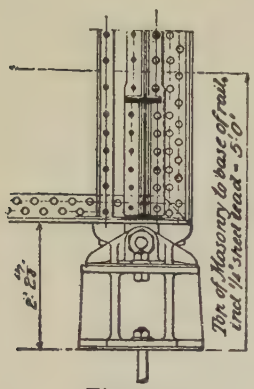


Fig. 13.

The pins were the most interesting part of the connection, and consisted of solid steel bars finished to a cross-section resembling two tangent 4-in. circles with their circumferences connected by straight lines about 3 in. apart, parallel to the line through the centers of the circles. The ends of the pins were turned cylindrical and projected several inches beyond the nuts to form shoulders long enough to afford bearings by which the girder could be raised if necessary. Oval washers between the nuts and the shoe and pedestal webs protected

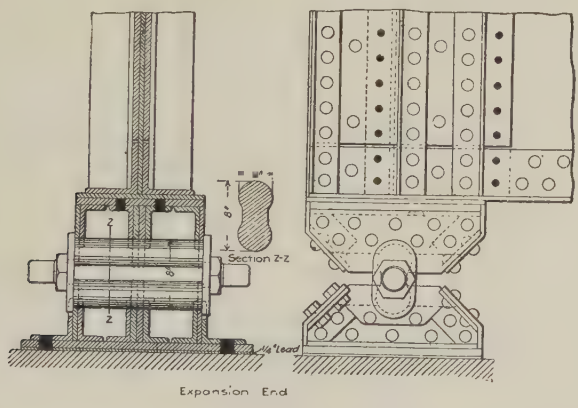
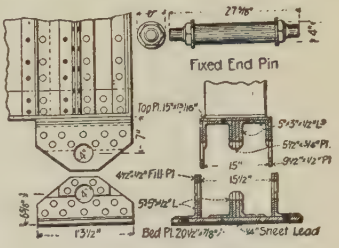


Fig. 14.

the ends of the pins and locked the shoe and pedestal together without requiring nuts of very large diameter.

The fixed end bearings (Fig. 15) for 72-ft. plate girders of the 1895 standard closely resemble those above described for the expansion bearings except that they have 4-in. cylindrical pins with full-hole bearing in the outer webs, locking the shoe and pedestal together.

In a modified form of this bearing (Fig. 16) the shoe and pedestal webs are connected by transverse end vertical plates inclosing the center spaces so as to exclude most of the dust and rubbish likely to accumulate there, and the pedestal is seated on a cellular cast-iron bed plate. The pedestal base and the top of the casting are planed, and the combination may be used either at the expansion end or at the fixed end to build up the height equal to that at the opposite end and obviate any difference in the levels of the masonry.



Fixed End Pin Bearing.

Fig. 15.

The standards of 1896 provided for the expansion ends of plate girder spans of 70 ft. and over (Fig. 17), similar riveted shoes with pin bearings in riveted pedestals seated on roller nests with riveted steel bed plates and sheet lead bearings.

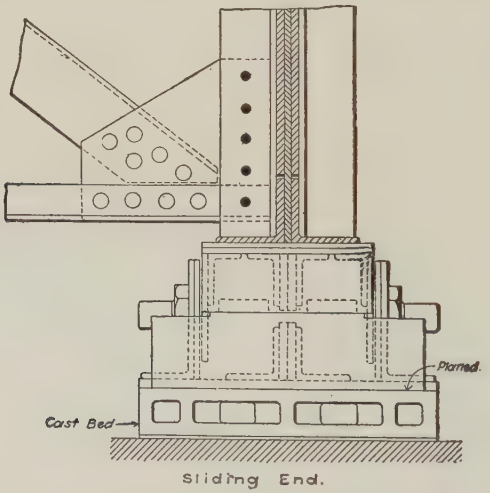


Fig. 16.

The shoes were made with three reinforced longitudinal vertical webs, the center one being in the plane of the girder web and having a bearing of about 140 degrees on the large pin which engaged full holes in the side webs of both shoe and pedestal, thus locking them together. The ends of the pins projected far enough beyond the shoes to afford connections for hydraulic

jacks, if it should become necessary to lift the girders for future changes or repairs.

The pedestals were practically inverted shoes with their base plates wide enough to project beyond the ends of the roller nests and engage the anchor bolts for which they had slotted holes. The rolled steel bed plates and the base plates of the pedestals both had planed bearing surfaces without grooves, and the rollers had no grooves for longitudinal guides. Transverse displacement was prevented by longitudinal angles riveted to the pedestal base and to the bed plate in the same vertical plane, with their vertical flanges planed to clear the roller necks and form a guard to exclude as far as possible the dirt and rubbish from

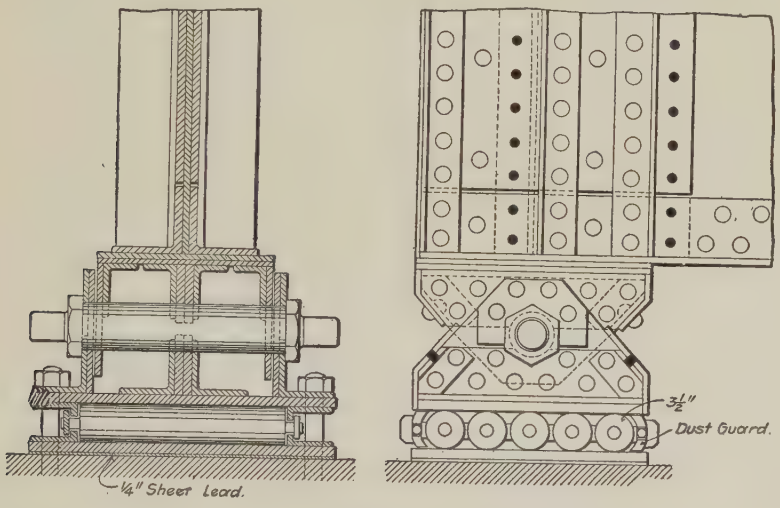


Fig. 17.

the interstices between the rollers. The sides of the roller nests were protected by curved transverse bars bolted to the end roller strips and fitted as close as possible between the pedestal base and bed plate, and moving back and forth with the rollers. The spaces between the shoe and pedestal webs were closed by transverse inclined plates riveted to them with angle clips.

Union Pacific Railroad.

Standard bearings adopted by the Union Pacific Railroad for deck single-track plate girders 30 ft. long have ¾-in. sole plates 14 in. wide and 16 in. long riveted to their 6 × 6-in. flange angles,

and are seated on unplaned 18 × ¾ × 18-in. bed plates. A pair of 1¼-in. rag bolts 12 in. long pass through holes in the flange angles at both ends, those at one end being 1⅜ in. in diameter, at the other being slotted to 1⅜ × 2½ in. The bearings for the 40-ft. single-track girders are the same, except that the sole and bed plates are both 16 × ¾ × 24 in. and the expansion slots are 3 in. long. The bearings for the 50-ft. girders are the same, except that the sole plates are 17 × ¾ × 24 in. and the bed plates are 19 × ¾ × 24 in.

The 60-ft. single-track deck girders have ¾-in. sole plates the exact width of the lower flanges, with $\frac{1}{16}$-in. clearance between each planed edge and the guide ribs on the upper surfaces of the cast-steel pedestals 8 in. high, which increase the bearing area on the masonry nearly 140 per cent above that of the sole plate. The girders are bolted to the pedestals by four 1-in. bolts, and the pedestals each have four 1¼-in. rag bolts. The metal in the pedestals is 1 in. thick uniformly, and they have four transverse and three longitudinal vertical webs dividing the space under the sole plate into six 4½ × 5¾-in. cells, accessible through 3 × 3-in. holes in the centers of the transverse webs.

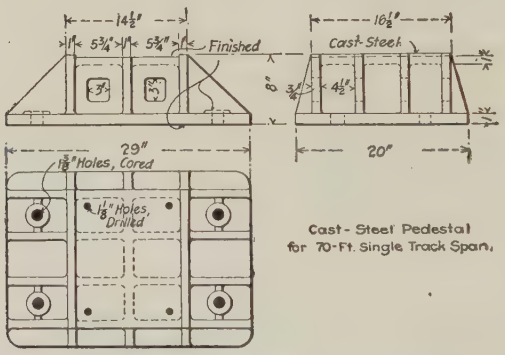
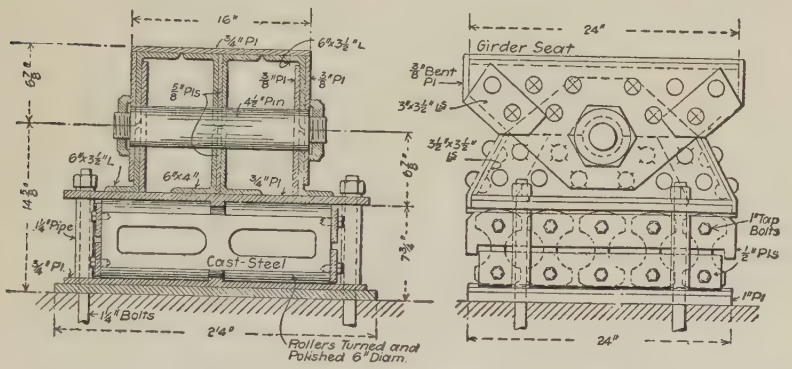


Fig. 18.

The bearings for the single-track deck girders 70 ft. long are like those for the 60-ft. girders, except that the pedestal bases have their dimensions increased to 29 in. wide and 20 in. long and have their corners rounded.

Spans of 80 ft. and over have pin bearings with shoes supported on rollers at the expansion ends and on fixed pedestals at the opposite ends. Through spans of 65 ft. and over have pin bearings with their shoes supported on rollers and pedestals of different types from those used for the deck spans.

Standard single-track deck girders, 80, 90, and 100 ft. long, have bearings which differ only in the dimensions. Shoes of very nearly the same width as the uniform bottom flange and of the same length and depth, but varying in the thicknesses of the materials and the sizes of the pinholes, are bolted, without sole plates, to the lower flange angles of the girders. They have three webs with the connecting angles of the outer ones turned inwards, and are made with full pin bearings on the web angles which reach to the center line of the pin. The pin bearings are half holes except in the outside web plates, which have full holes. The roller end pedestals correspond to the shoes, reversed, except that the connection angles for the outer webs have their flanges turned outwards, and the base plate



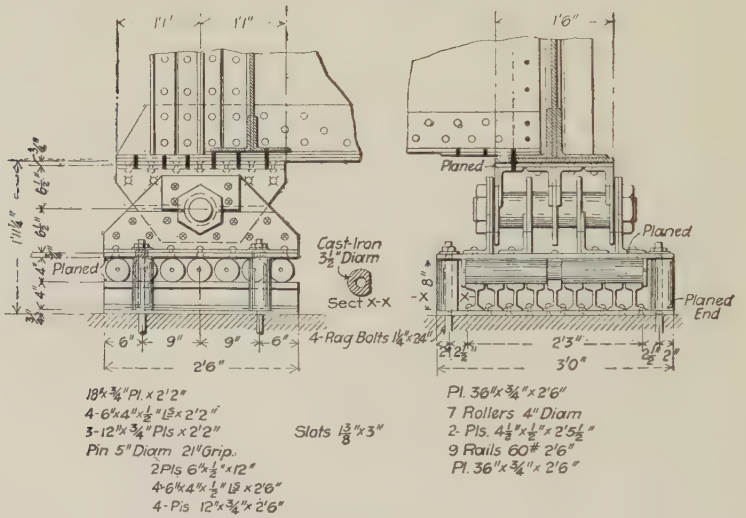
Expansion Bearing for 80-Foot Deck Span

Fig. 19.

projects several inches beyond them to engage the anchor bolts clear of the angles. The webs are in the same vertical planes in shoes and pedestals, so that the pin bearing is reduced to direct compression, and shear and bending are eliminated. The outside web plates are riveted on opposite sides of their connection angles so as to match past each other, and are locked together by the pin with shouldered ends and chambered nuts. The inclined edges of all web plates are connected by cover plates, which completely inclose the interior of the shoe and pedestal so as to exclude rubbish. At the expansion end the base plate of the pedestal is seated on five cast-steel segmental rollers, and has riveted to it a center longitudinal rib to engage a corresponding groove in them. The sides of the rollers are hollowed, and their ends are tap-bolted top and bottom, to side

plates arranged with end stops to lock them and prevent the rollers from having an angular displacement greater than the allowed maximum. The rollers are seated on a bed made with two rolled plates riveted together and having a slightly larger area than the pedestal base. Four $1\frac{1}{4}$-in. rag bolts pass through $1\frac{3}{8}$-in. round holes in the bed plate and 2 × 3-in. slotted holes in the pedestal base plate. Between the bed plate and pedestal the anchor bolts have pipe separators.

Through double-track girders 65 to 95 ft. long have pin bearings with fixed and rolling pedestals. The shoes and pedestals are similar to those for the deck girders, except that all



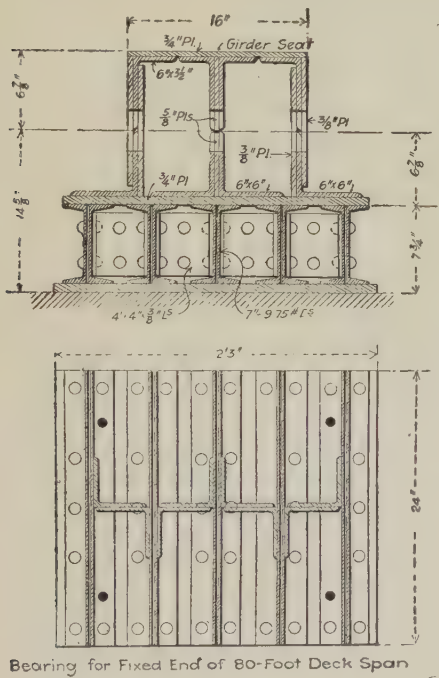
Expansion Bearing for 95-Ft. Span

Fig. 20.

their webs have full holes for the pins, and that the pedestal has four webs alternated with those of the shoe, so that the pin is subjected to bending stresses. At the expansion end the shoes are seated on nests of full-cylinder rollers connected by tap bolts with side strips planed on the upper edge to fit closely against the under surface of the pedestal base plate and having the lower edge project below the top of the roller bed, thus forming a dust guard to protect the roller surfaces. The roller bed is made with longitudinal track rails having their heads planed horizontal for roller bearings, and having one side of each head and flange partly cut off, so that they can be seated closer

together on the base plate to which they are riveted. The longitudinal edges of the base plates of both the roller bed and the pedestal project several inches beyond the ends of the rollers in order to receive the four anchor bolts which pass through vertical cast-iron sleeves having both ends planed square for bearing against the opposite surfaces of the plates. At the fixed end of the span the rollers and their beds are omitted, and in their place there are cellular cast-iron bolsters made of metal $1\frac{1}{4}$ in. thick, planed top and bottom.

The fixed end pedestals correspond to the expansion end pedestals except that the base plate forms the top cover plate of a bolster of a height to compensate for the rollers and maintain both ends of the girder at the same height above the masonry seats. The bolster is made with five longitudinal webs, each composed of two 7-in. channels, back to back, riveted to a pair of bottom plates slightly larger than the top plate. Short vertical angles with their flanges turned in opposite directions are riveted to the channel webs on the center line, and their outstanding flanges are riveted together to form a transverse diaphragm bracing all webs. The four anchor bolts pass through the bolster, and their nuts take bearing on the pedestal base plate.



Bearing for Fixed End of 80-Foot Deck Span

Fig. 21.

Canadian Pacific Railroad.

Standard short-span girder bridges on the Canadian Pacific Railroad have bearings at both ends on shoe plates riveted to the lower flange angles and seated on steel bed plates bolted to the masonry with single $1\frac{1}{2}$-in. anchor bolts 38 in. long, set with

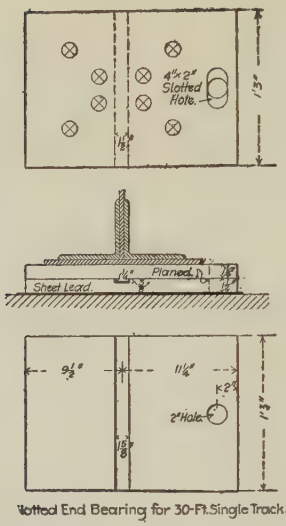
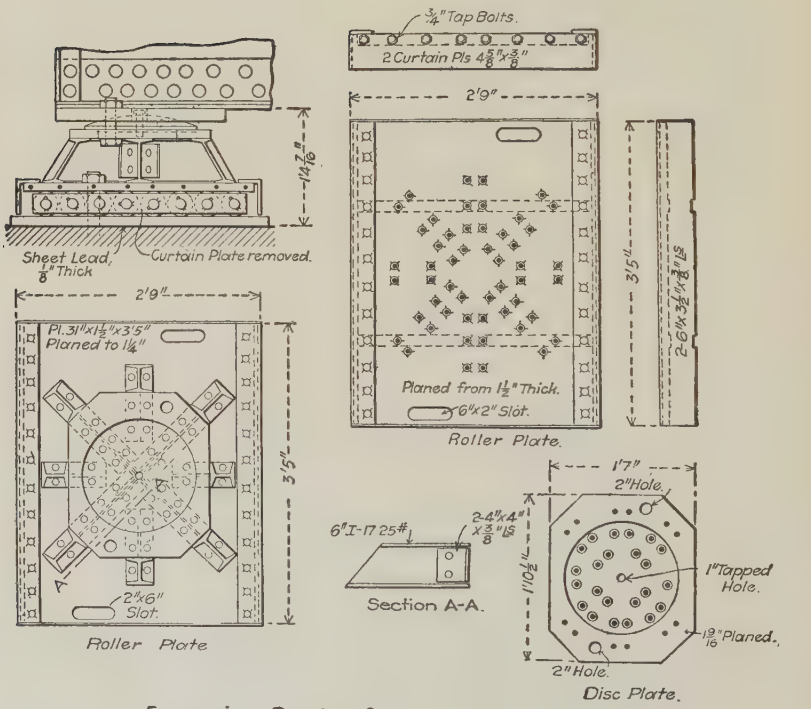


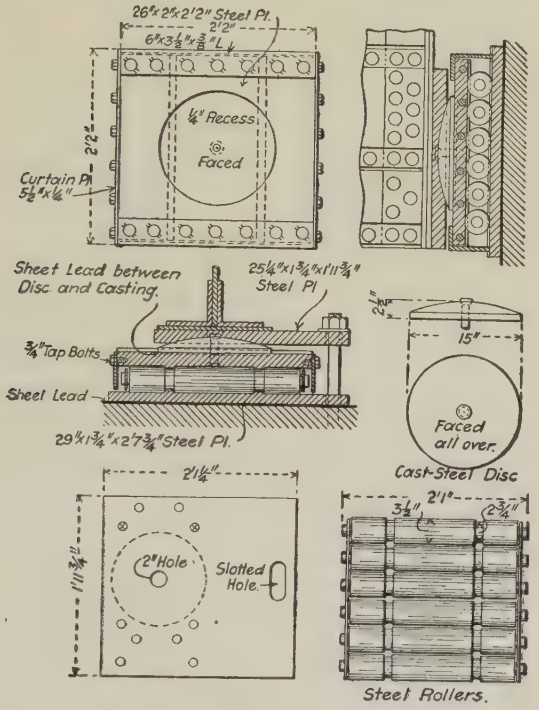
Fig. 22.



Expansion Bearing for 100-Foot Through Span.

Fig. 23.

thin pure cement grout poured into the $1\frac{3}{4}$-in. holes before the bolts are inserted. For a clear span of 30 ft. the girders are 33 ft. long, $36\frac{1}{2}$ in. deep, and 15 in. wide, with sole plates $20\frac{3}{4}$ in. wide, 15 in. long, and $1\frac{1}{4}$ in. thick. They are made from $1\frac{1}{2}$-in. plates planed on the under side to leave a $1\frac{3}{8} \times \frac{1}{4}$-in. center longitudinal rib fitting a corresponding $1\frac{5}{8} \times \frac{1}{8}$-in. groove planed in the bed plate of the same dimensions. The bed plates are riveted eccentrically to the girder flanges to project 2 in. farther



Expansion End Bearing for 60-Foot Half Deck Span.

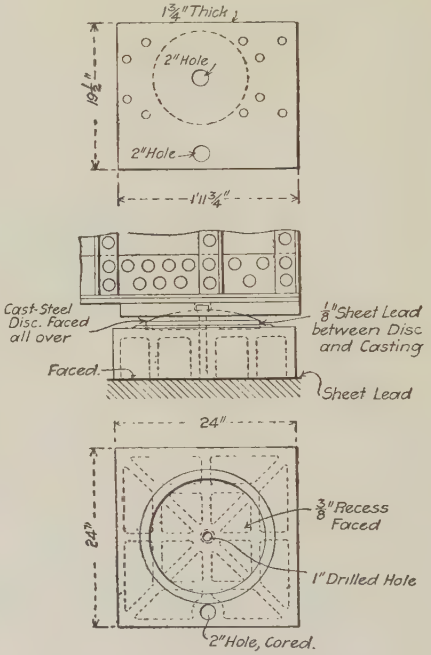
Fig. 24.

beyond the outer than the inner edge, and have there one anchor bolt hole 2 in. in diameter for the fixed ends and slotted 2×4 in. in the expansion ends. The bed plates are faced on the upper surface only and have corresponding 2-in. bolt holes. They are seated on $15 \times \frac{1}{8} \times 22\frac{3}{4}$-in. sheets of soft lead.

Long-span plate girders of the latest standard design for the Canadian Pacific Railroad have spherical bearings at both ends to provide for deflection, distortion, and any irregularities of

construction or substructure, and to give perfect and uniform reactions under all conditions. For a 60-ft. clear span the girders are 15 in. wide, 72 in. deep, and 65 ft. 4 in. long over all, and have bearings 63 ft. 1 in. apart on centers. At each end a $1\frac{3}{4}$-in. rolled steel sole plate $23\frac{3}{4}$ in. wide and $19\frac{1}{2}$ in. long is riveted eccentrically to the lower flange cover plate, being flush with the inner edge and projecting $10\frac{1}{4}$ in. beyond the outer edge to engage a single anchor bolt clear of the flange angle and cover plate.

Symmetrically with the girder web, the plate is bored 1 in. deep on the under side to fit the convex surface of a horizontal cast-steel disk 15 in. in diameter, with the upper surface turned to a spherical segment with a radius of $19\frac{5}{8}$ in. It is $2\frac{1}{2}$ in. thick in the center and about 1 in. thick on the edges. The flat under surface of the segment is seated on a $14\frac{3}{4}$-in. disk of sheet lead $\frac{1}{8}$ in. thick, set in a hole $15\frac{1}{8}$ in. in diameter and $\frac{1}{4}$ in. deep in the top of a casting or plate, to which the disk is secured by a vertical 1-in. center rivet with a countersunk lower head and an upper head which has clearance in a 2-in. hole in the sole plate. When the girder is seated on the disk there is a clearance of $1\frac{1}{4}$ in. between the sole plate and the disk plate, sufficient for any deflection.



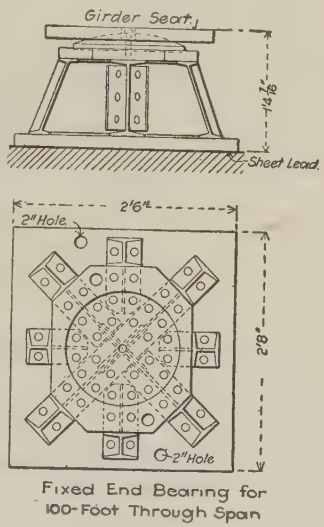
Fixed End Bearing for 60-Foot Half Deck Span

Fig. 25.

At the expansion end the disk is seated on a rolled-steel plate 2 in. thick, with the under side planed to $1\frac{3}{4}$ in. thick, leaving two $\frac{7}{8} \times \frac{1}{4}$-in. longitudinal ribs which engage grooves in the six $3\frac{1}{2}$-in. rollers. The rollers are set on a base plate corresponding to the upper plate, but projecting $5\frac{3}{4}$ in. beyond its outer edge to receive a single $1\frac{1}{2}$-in. anchor bolt clear

of the roller nest and upper roller-bearing plate which engages a slotted hole 4½ in. long in the sole plate. The bed plate is seated on a 31 × ⅛ × 34-in. piece of sheet lead. The space between the upper and lower roller-bearing plates is inclosed by vertical dust-guard pieces to prevent the accumulation of dirt or rubbish around the rollers. These are made with two angles riveted across the front and rear ends of the top plate parallel with the rollers, and with two thin longitudinal vertical plates tap-bolted to the edges of the top plates, all of them having their lower edges planed to a clearance of $\frac{1}{16}$ in. with the base plate.

For a 100-ft. clear span the girders are 8½ ft. deep, 19 in. wide, and 105 ft. 10½ in. long, with bearings 103 ft. 1½ in. apart. The bearings are of the same type as those described for the 60-ft. span, with which the details correspond, except that each has two anchor bolts and that the disks are seated on riveted-steel pedestals. The pedestals have thick horizontal top and bottom plates connected by eight longitudinal, transverse and diagonal I-beam webs. The disk is tap-bolted to the top plate in which it is seated, and the I-beam webs are connected at the center by short vertical angles. The sole plate and the pedestal cap plate are connected by two 1½-in. vertical bolts, and the I-beams are 6 in. deep. At the expansion end the base plate of the pedestal has two slotted holes 6 in. long for the anchor bolts.

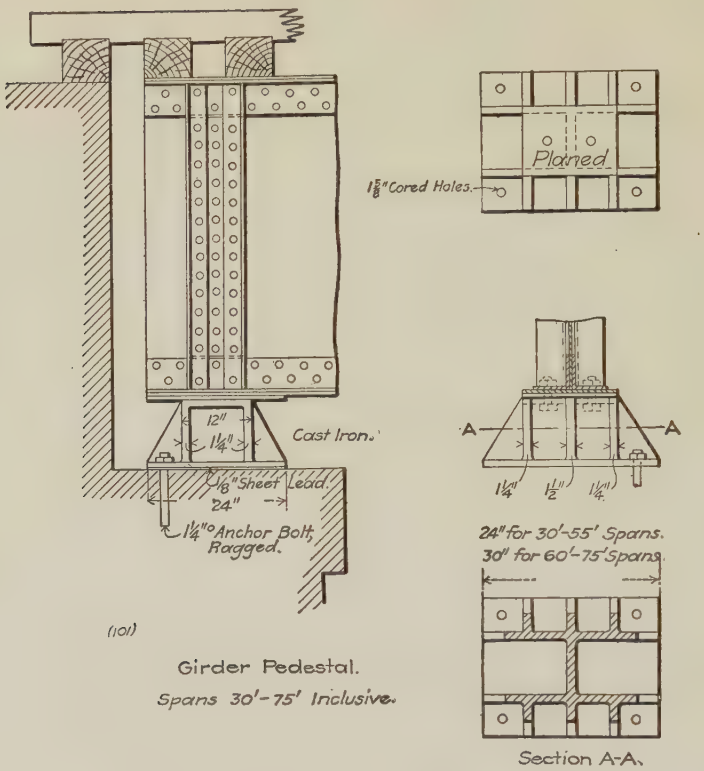


Fixed End Bearing for 100-Foot Through Span

Fig. 26.

For less than 60-ft. clear spans the fixed end of the masonry seat is at the same level as at the expansion end, and the omission of the rollers is compensated for by a cast-iron pedestal 6¾ in. high, which receives the disk and replaces rollers, roller bearing and bed plate. The casting has longitudinal transverse and diagonal vertical webs, and is secured to the masonry.

In a 100-ft. clear span the pedestal is made with 6-in. I-beams at the expansion end, but at the fixed end the I-beams are 10 in. deep to compensate for the omission of the 3½-in. rollers, and there are 2-in. round holes for the anchor bolts, and the 35 × 1½

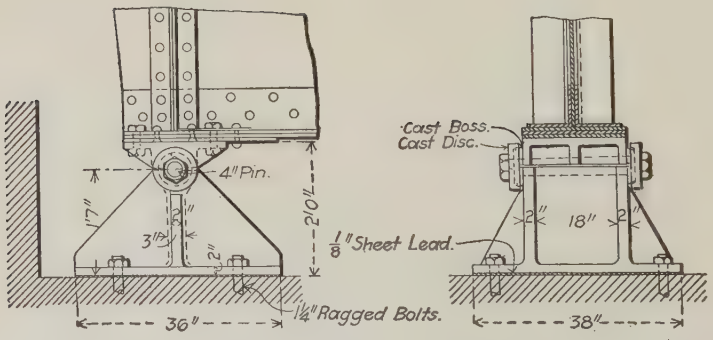


(101)

Girder Pedestal.

Spans 30'-75' Inclusive.

Fig. 27.



Standard Fixed Shoe.

Fig. 28.

× 42-in. bed plate is omitted, and the pedestal is seated directly on the 32 × ⅛ × 34-in. piece of sheet lead.

Baltimore & Ohio Railroad Standards.

Standard short-span plate girders on the Baltimore & Ohio Railroad have double pairs of vertical angles and filler plates close together to reinforce the ends of the webs for maximum shear, and transmit the pressure to the lower flange, which has a sole plate riveted to it, symmetrical with the web angles. The sole plate is bolted to a planed cast-iron pedestal seated on a lead plate and anchor-bolted to the substructure masonry. The cap of the cast-iron pedestal just covers the ends of the reinforcement angles, and with them is a few inches clear of the end of the

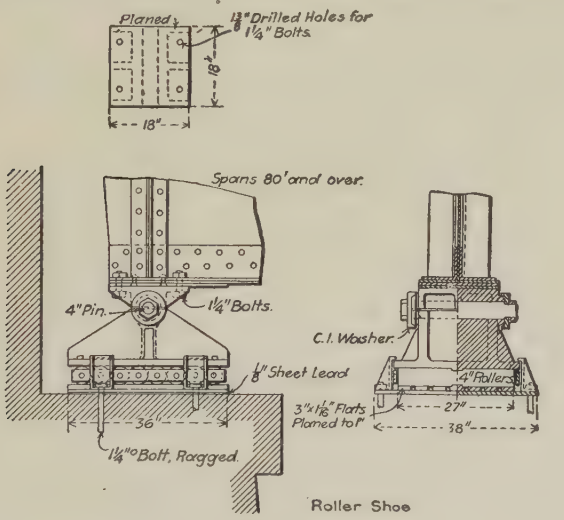


Fig. 29.

girder web. The base is extended symmetrically to be both wider and longer than the top and secure the desired bearing area. It is connected to the top plate by longitudinal and transverse vertical webs, the former being in the plane of the girder web. For spans of less than 80 ft. this bearing is used under both ends, the only difference between the fixed and the expansion ends being that the bolt holes through the sole plate are round in the former and slotted in the latter case.

The standard roller bearing adopted on recent work, for spans of 80 ft. or more, is made with a cast-steel shoe and pedestal and a riveted steel bed plate. The shoe is bolted to a sole

plate on the lower flange of the girder, and has a full-length vertical transverse web reinforcing the semi-cylindrical bored bearing for the upper side of the 4-in. steel pin. This bearing is also stiffened by one center and two side longitudinal webs. The pedestal is essentially a reversed shoe, but is higher, with a base extended both longitudinally and transversely, having longitudinal and transverse vertical webs or diaphragms 2 in. thick. A finished rib on the center line of the planed under surface of the base engages the eight 4-in. rollers. Bosses on the outside of the shoe and pedestal webs engage chambered disks that fit over the ends of the pin and are secured by nuts on its shouldered ends. The bed plate has a finished thickness of 2 in. and is made with 6 × 1-in. bearing strips 1 in. apart in the clear. It projects 5½ in. beyond each end of the rollers to receive four 1¼-in. anchor bolts, which also secure the bottoms of flanged cast-steel pieces of Z-shaped cross-section, which have

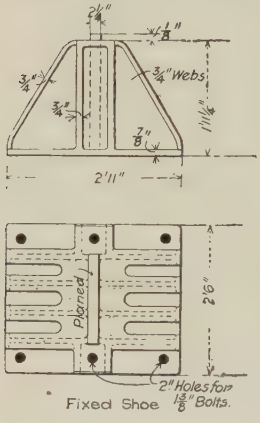
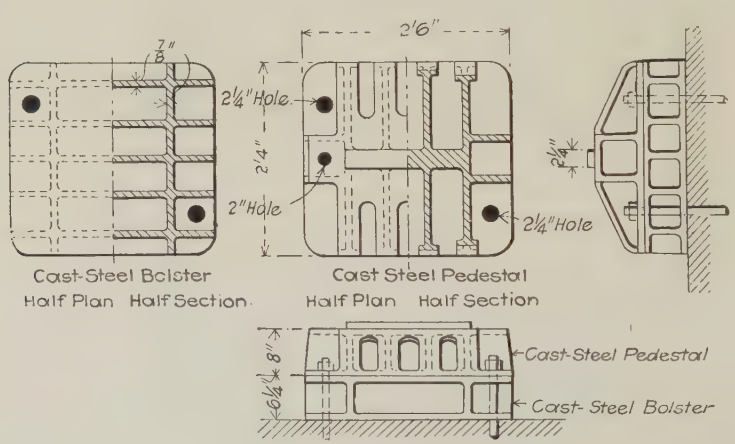


Fig. 30.



Fixed End Bearing

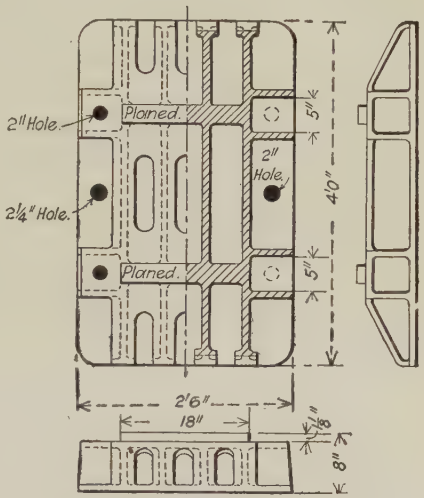
Fig. 31.

upper flanges engaging the base plate of the pedestal. Cast stops on the pedestal limit its motion each side of the clamps. The bed plate is set on a thin sheet of lead, and the ends of the

rollers are shouldered into side strips, which are secured by cotters passing through the projecting ends of the long shoulders on the outside rollers.

Chicago Great Western Railway Standards.

Standard bearings of the deck and through spans of the plate girder bridges of the Chicago Great Western Railway are made with sole plates riveted to the bottom flanges, which are reinforced there by pairs of vertical web-stiffener angles. They are seated on planed transverse ribs about $1\frac{1}{8}$ in. high and 2 to $2\frac{1}{2}$ in. wide on top of cast-steel pedestals with transverse and horizontal webs and top and bottom plates. . For short spans these pedestals are alike at both ends, except that at the expansion ends they have slotted holes for the anchor bolts through them and the girder flanges and a $\frac{1}{2}$-in. masonry plate. Where two spans are supported on the same pier, the pedestals are double, and each is made with two seats.



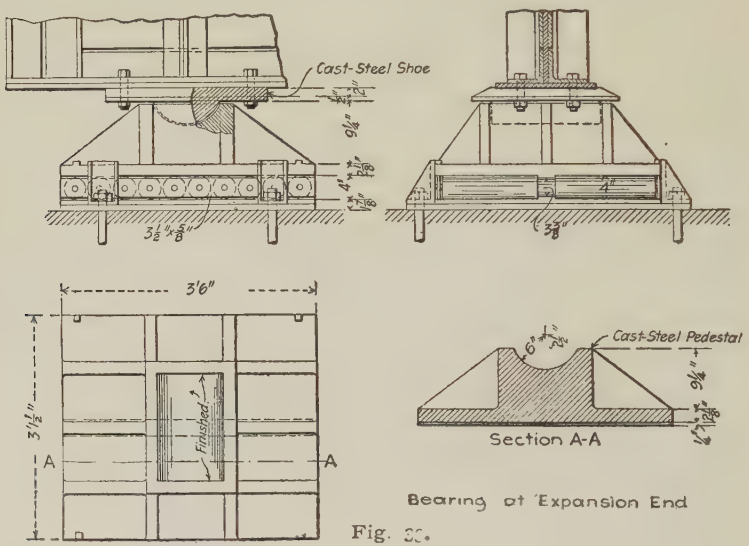
Two Seat, Fixed Pedestal.
Fig. 32.

For long spans the expansion end pedestals have longitudinal guide ribs on the lower surface and engage nests of cylindrical rollers on cast-steel bed plates with longitudinal grooves and side guide ribs on the upper surface. The roller frames are made with cast-steel longitudinal pieces and transverse angles all deep enough to nearly close the space between the pedestals and the bed plate, and thus protect the roller nest from dirt, snow, etc. The pedestals are bolted to the girder flanges and the bed plates are anchor-bolted to the masonry by two diagonally opposite $1\frac{3}{4}$-in. bolts 3 ft. long. In some cases the fixed end pedestals are identical with those at the expansion ends and are seated on cast-steel bolsters just high enough to compensate for the combined height of the rollers and bed plate and so keep

the masonry at the same elevation at both ends of the span. In other cases the fixed end pedestal is made high enough to dispense with the bolster.

Wabash Railroad Standards.

Standard bearings for long-span plate girders on the Wabash Railroad are illustrated by those for a single-track skew through bridge of two 85-ft. spans on the Toledo division over One Leg Creek. The bridge is proportioned for a live load consisting of a train weighing 5,000 lb. per lin. ft., preceded by two coupled locomotives each with 50,000 lb. on each of four axles, 5 ft.



Bearing at Expansion End

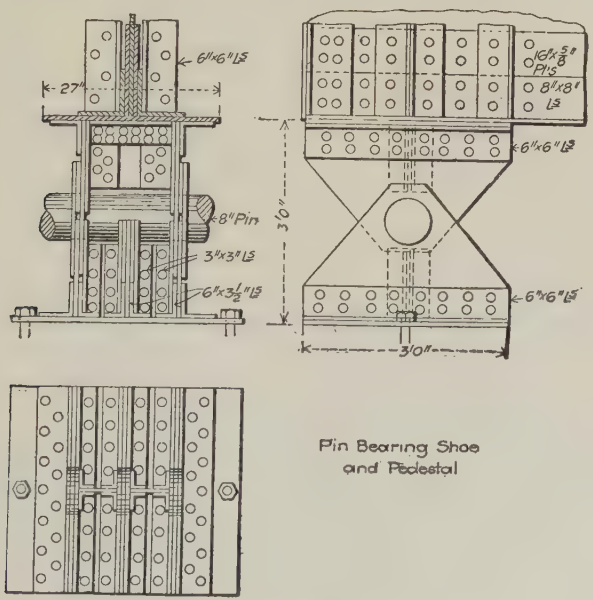
Fig. 30.

apart. One girder is designed for future service as the center girder of a double-track bridge, and its bearings are cylindrical to provide for deflections or inequalities of setting, and are fixed for both spans on their common center pier, and have expansion provision on both abutments. An 18 × 27-in. sole plate is riveted to the lower flange of the girder, and to it is bolted a cast-steel bearing plate 24 in. wide and 2 in. thick, on the bottom of which is a finished segment of a transverse cylinder 12 in. in diameter and 18 in. long with a versed sine of 4 in. This segment engages a corresponding recess with a versed sine of $2\frac{1}{2}$ in. in the top of a cast-steel pedestal with its base extended to $37\frac{1}{2} \times 42$ in. and stiffened by vertical transverse and longitudinal

ribs. The space between the inner sides of these webs is solid, thus forming a heavy block to receive the girder load. The pedestal is seated on a nest of ten 4-in. rollers of the usual type with a guide strip in the center. The rollers are seated on a $46\frac{1}{2} \times 42$-in. bed plate 1 in. thick, with nine longitudinal 3-in. bearing strips riveted to the upper surface. The ends of the roller are inclosed by side strips $3\frac{1}{2}$ in. wide, but there is no provision for inclosing the sides of the roller nest. The longitudinal movement of the pedestal is limited by a pair of clamps on each side, with which are cast-steel lugs, Z-shape in cross-section, held down by the anchor bolts through the outer edges of their base plates. They are set on the bed plates, clear of the rollers, and their upper flanges project over the base plate of the pedestal and have about 2 in. clearance from small vertical projections on the base plate which serve as stops. The fixed end bearing is the same as that of the expansion, and except that the rollers and bed plate are omitted and the pedestal is made correspondingly higher.

New York Central Railroad Standards.

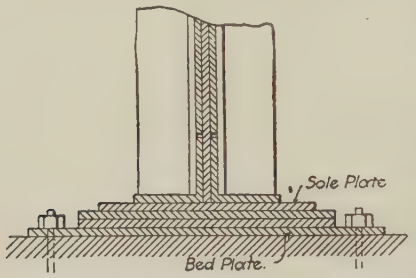
Standard girders 75 feet or more in length on the New York Central & Hudson River Railroad have bearings at both ends



Pin Bearing Shoe and Pedestal

Fig. 34.

on pins from 4 to 6 in. in diameter, which engage riveted shoes and pedestals, both of which correspond essentially to those commonly used for short-span bridge trusses. The shoes have top plates of the same width as the girder flanges, to which they are field-riveted, and generally have three webs made up with plates and angles. The middle web has a half-hole pin bearing, and the side webs have full-hole pin bearings. The pedestal has also three webs, but the base plate is wider than that of the shoe, and overhangs the ends of the rollers, so that the anchor bolts through its $1\frac{1}{2} \times 2\frac{1}{2}$-in. slotted holes clear the rollers. Its outer webs clear the side webs of the shoes. The base plate is made double, with the edges of the smaller bottom piece planed to a width exactly equal to the lengths of the rollers. The bed plate is a webbed casting about 6 inches high, with a longitudinally grooved roller surface. Guide ribs are planed out of the solid in the center line of both the pedestal base and the top of the bed plate, and the square ends of the rollers have necks engaging the side strips of the roller frames. The latter are made $\frac{1}{2}$ in. wider than the diameter of the rollers, so that they overlap the bearing surfaces and form a close dust guard inclosing the roller nest. These strips are secured together by shouldered-end bolts parallel to the rollers. The cast bed plates are seated on 1-in. rolled steel masonry plates with $\frac{1}{4}$-in. sheet lead fillers under them.



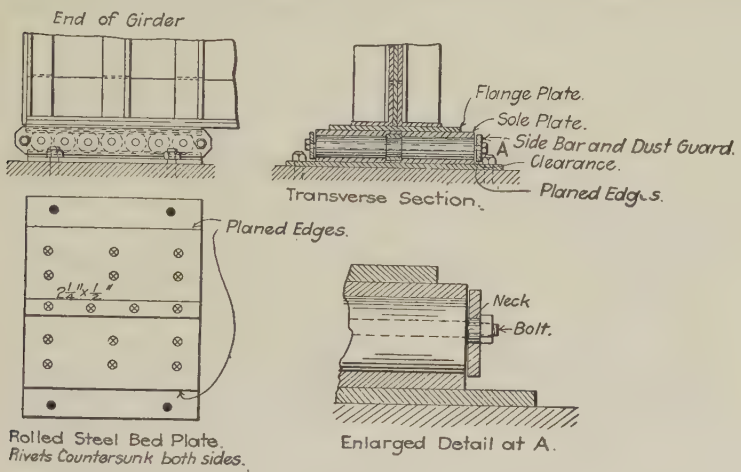
Reinforced Plates at Fixed End Bearing

Fig. 35.

The fixed end bearings correspond to those for the expansion ends, except that the pedestals are unplaned, have both base plates of the same size, and have no guide ribs. They also are seated on $\frac{1}{4}$-in. sheet lead fillers $\frac{1}{2}$ in. wider and longer than their own dimensions, and are secured by four $1\frac{1}{4}$-in. anchor bolts through $1\frac{3}{8}$-in. round holes. The pedestals are often of the same heights as at the fixed and expansion ends, and the masonry seats are built a few inches higher to compensate for the absence of roller nests and cast bed plates.

Erie Railroad Standards.

An example, typical of the simplest type of roller bearing for moderate length spans of both railroad and highway bridges, is the standard expansion bearing or short-span bridges adopted by the Erie Railroad. It would be the most common except for center guide strap, which is expensive and seldom used by bridge companies until they are reminded of the requirement. It is an essential feature, however, and keeps the rollers from skewing out of line. According to the present standard



Expansion Bearing for Short Span Girders

Fig. 36.

designs, short-span plate girders, or those which are so proportioned and loaded that the deflections are not important enough to cause serious distortion, are seated at both ends on flat sole plates riveted to the lower flanges. The plates are usually about 1 in. thick and 3 or 4 in. wider than the flange cover plate, and are long enough to correspond with the required number of rollers, the amount of projection being determined by their thickness. The thickness of the sole plate is proportioned to reduce the maximum pressure per linear inch on rollers to 300 lb. × diameter in inches and the thickness of the bed plate to reduce the maximum pressure on the masonry to 250 lb. per square in. on the masonry. The stone pedestals are all dressed very accurately to the level of the lowest point in case of an inclined grade, and are made true enough to receive the bed plate or pedestal without grouting, mortar, or lead filler.

In calculating the bending stress in the projecting edges of the bed plate if the latter is assumed to be made of parallel longitudinal strips 1 in. wide and 1 in. thick, receiving a vertical pressure of 250 lb. per lin. in., and the maximum fiber stress is allowed to be 12,000 lb. per square in., the formula, $\frac{bh^2}{6} S = \Sigma M$, becomes, upon substituting the given and assumed values, $\frac{1 \times 1 \times 12{,}000}{6} = 250\, d \times \frac{d}{2}$ where d is the distance in inches which each side of the sole plate projects beyond the girder flange. Solving, $d = 4$ in., the maximum projection for a 1-in. plate. If two or more plates are riveted together, as is sometimes done for very heavy loads, the projection for the lowest

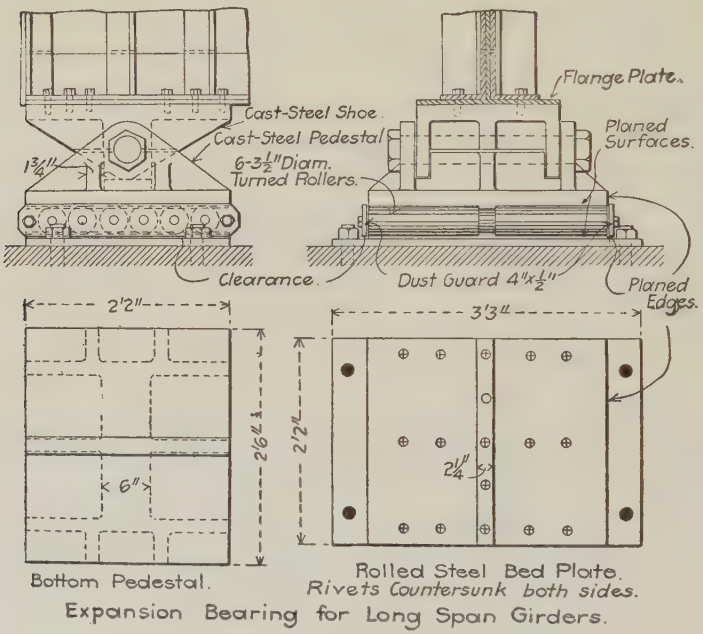
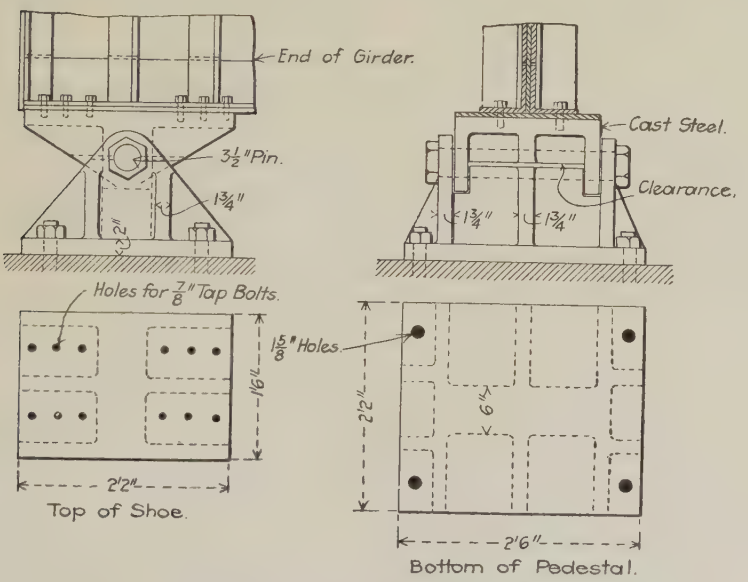


Fig. 37.

and widest plate would be first found in this manner and the successive upper ones in the same way, modifying the formula to correspond with the changed values of h, the thickness of the plate. Rollers are made with a minimum diameter of 3 in., and their lengths and numbers are proportioned to conform with the loading and with the restrictions, if any, in the dimen-

sions of the bridge seats, which if very narrow may necessitate longer rollers than would otherwise be used.

The longitudinal edges of the plates are planed true, and are in the same vertical planes with the shoulders of the rollers and with the upper plates of the double bed plates, which are also planed true. The longitudinal vertical bars forming the side strips of the roller frames are made $\frac{1}{2}$ in. wider than the diameter of the rollers, receive the necks of the latter, and are secured together in the usual way by transverse bolts and thimble pipes at the ends and also at intermediate points for larger nests of rollers. By this arrangement the side strips overlap the edges of the top and bottom plates with clearance above the base plate, and thus form well-sealed dust guards. They do not require any difficult fitting, are not liable to derangement, and permit free longitudinal motion. The bed plates are made with a 1-in. and a $\frac{3}{4}$-in. steel plate riveted together with double-countersunk rivets, and have a center longitudinal $2\frac{1}{4} \times 1\frac{7}{16}$-in. rib recessed $\frac{3}{16}$ in. into the upper side to guide the notched rollers.



Fixed Bearing for Long Span Girders.
Fig. 38.

When the girders have long spans, or where in shorter spans the deflection distortion is likely to be enough to cause eccentric

stresses in the flat bearings, or where the load will for some reason be very heavy, the girders are seated at both ends on pin bearings. According to the specifications the expansion ends of girders less than 60 ft. long shall have sliding bearings, those over 60 ft. long shall have roller bearings, and spans of 90 ft. and over shall have pin bearings, but in practice these limits are greatly reduced. In long spans, care is taken to secure bearing between the girder web and the sole plate or end of the lower flange cover plate.

A cast-steel shoe with a single transverse and triple longitudinal webs is riveted to each end of the lower flange. The lower edge of the transverse web is thickened and bored to a 180 degree full-length bearing on the $2\frac{1}{2}$-in. hinge pin which engages full holes in the deeper side longitudinal webs. The pin is seated in three transverse web bearings in a cast-steel pedestal, which is virtually a reversed shoe, with the base extended longitudinally and transversely far enough to afford a proper distribution of the load and diminish the unit stresses to the required amount per linear inch of the rollers. The pedestal is notched to receive the side longitudinal ribs of the shoe, to which it is locked by the pin passing through the full holes in the side webs. The expansion end pedestal is seated on a nest of rollers with their ends turned down to a diameter of 1 in. to engage the $4 \times \frac{1}{2}$-in. side bars which form dust guards overlapping the planed edges of the base plate and the bed plate, as described for the expansion ends of the short-span girders of the Erie Standards. The bed plates for both long and short spans are secured to the masonry by four $1\frac{1}{4}$-in. bolts, 15 in. long, with the lower ends split 4 in. and wedged.

Atchison, Topeka & Santa Fe Railway Standards.

The bridges of the Atchison, Topeka & Santa Fe Railway are standard design plate girders for most spans of from 26 to 106 ft. All of them have bearings at both ends on sole plates and solid or riveted bed plates, and all are anchored at one end and slide at the other end. Spans of 60 ft. and under have flat sole plates riveted to their lower flanges and projecting about an inch beyond them on each edge. They are seated on bed plates from $\frac{3}{4}$ in. to 1 in. thick, and about 2 in. wider than the sole plates. The bed plates on piers are long enough to receive the ends of both adjacent girders, and at the abutments

are of the same lengths as the sole plates. The girders are anchored to the masonry at each end by two wedge bolts, which pass through slotted holes at the expansion end.

Girders of 70-ft. span and over have sole plates $\frac{7}{8}$ in. thick, with both ends beveled to concentrate the bearing on the center third. These take bearing on pedestals made with longitudinal Z-bars riveted between top and bottom plates. The sole plates are secured to the top plates of the pedestals by three vertical

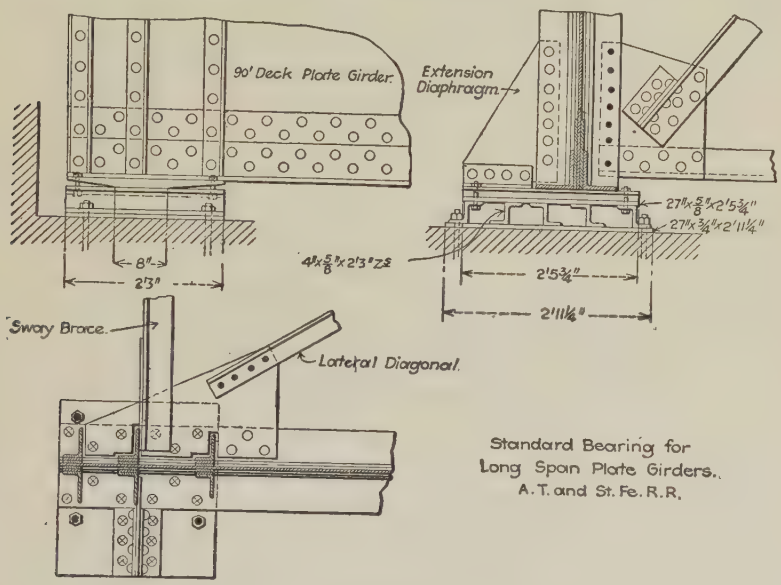


Fig. 39.

1-in. bolts with the nuts slack enough to allow a slight vertical displacement when the girders deflect on the rocker plates. The fourth bolt is usually omitted on account of the interference of the end lateral connection plate, which engages the inside of the lower flange of the girder just above the sole plate. The base of the pedestal is wider than the top by twice the width of the Z-bar flange, and it is secured to the masonry by four 1-in. wedge bolts through the base plate only.

Where the girders are very deep, greater stability is afforded to their bearings by extending the sole plate beyond the outer flange of the girder and stiffening it with a transverse diaphragm riveted to the end vertical web-stiffener angle, the pedestal being of course made with corresponding dimensions.

In all spans both fixed and expansion end bearings are exactly alike, except that in the latter the sole plates have slotted holes for the pedestal and bed-plate bolts, and in the former they have round holes.

Grand Trunk Railroad Standards.

The standard plate-girder bearings adopted by the Grand Trunk Railroad Company are illustrated by the details at the ends of an 87½-ft. deck span at mile post 34½ of the fifth district. At the expansion ends heavy sole plates are riveted to the bottom flange and project about 8 in. beyond the edges of the

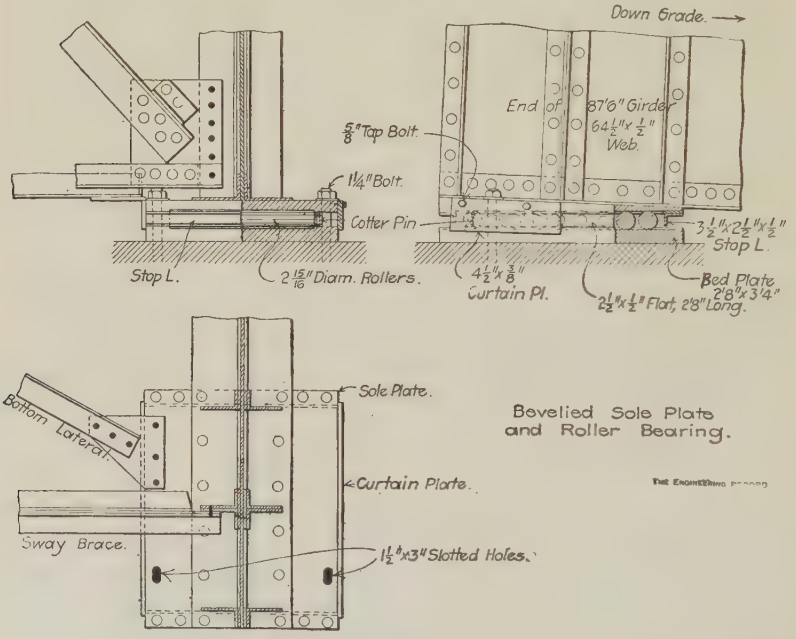

Down Grade.
End of 87'6" Girder
64 1/2" x 1/2" Web.
5/8" Top Bolt.
1 1/4" Bolt.
Cotter Pin
3 1/2" x 2 1/2" x 1/2" Stop L.
Stop L.
2 15/16" Diam. Rollers.
4 1/2" x 3/8" Curtain Pl.
Bed Plate 2'8" x 3'4"
2 1/2" x 1/2" Flat, 2'8" Long.
Sole Plate.
Bottom Lateral.
Curtain Plate.
Sway Brace.
1 1/2" x 3" Slotted Holes.
Bevelied Sole Plate and Roller Bearing.

Fig. 40.

cover plate on each side, giving room on the inner edge for the riveted connection to the lateral diagonal angle. As the bridge is on a grade the plate is beveled $\frac{1}{16}$ in. in 12 in. longitudinally to correspond, in order to secure uniform bearing on the horizontal abutment surface. The center of the plate is planed down to leave ⅛-in. longitudinal guide ribs on each side, between which the nest of nine $2\frac{5}{16}$-in. rollers 24 in. long are seated. The rollers have 1½-in. necks ¾ in. long, engaging the side

plates, through which three of them project and are secured by cotter pins. The rollers are secured longitudinally by transverse angles riveted to the ends of the sole plate, and their lateral displacement is prevented by longitudinal guide ribs planed on the upper surface of the bed plate to correspond to those on the sole plate. Vertical $4\frac{1}{2}$-in. cover plates are tap-bolted to the edges of the sole plates to inclose the roller nests and exclude the dust. The bed plates are anchored to the masonry by two short $1\frac{1}{4}$-in. bolts, which project through $3 \times 1\frac{1}{2}$-in. slotted holes in the sole plates.

CHAPTER VII.

STANDARD SPANS.

The Atchison, Topeka & Santa Fe Railway.

WITHIN the last few years an aggregate of from 10,000 to 15,000 tons a year of short-span bridges have been built for the lines of the Atchison, Topeka & Santa Fe Railway. They conform to a set of standard designs which include six general types for single-track structures up to spans of 106 ft., varying in length by increments of 3 and 5 ft. They are calculated with ample strength for the heaviest locomotives and train loads now in use on this road, and thus provide for reasonable working strength and a certain amount of future increase in duty. They are designed to secure rigidity under high-speed traffic, and special care is taken to construct them from a small range of standard sizes of plates and shapes, so that material can easily be secured for them or be kept in stock. All girders and some deck spans are shipped whole, completely shop-riveted, and the connections are arranged so that all field holes can be filled and the field rivets can be driven to the best advantage. All deck spans are designed as single-track structures, and where there are two or more adjacent tracks at the same crossing, the superstructure consists of a corresponding number of single-track spans side by side.

All bridges are proportioned for two 139-ton consolidation engines coupled and followed by a uniformly distributed train load of 3,200 lb. per linear ft. Each engine has a 55-ft. wheel base with axle loads and spacing as follows, beginning with the front truck: 18,000 lb. 7½ ft., 44,000 lb. 4½ ft., 44,000 lb. 4½ ft., 44,000 lb. 4½ ft., 44,000 lb. 10½ ft., 20,000 lb. 5 ft., 20,000 lb. 5½ ft., 22,000 lb. 5 ft., 22,000 lb. 8 ft., the 8-ft. space being the distance between the last axle in the tender and the first axle in the forward truck of the next locomotive. Heavy grade spans are loaded 50 per cent heavier with two 208½-ton consolidation engines and a 4,800-lb. train load. All material is open-hearth steel, excepting the rivets, and has an ultimate strength of from 55,000 to 65,000 lb. per

square in., an elastic limit of 55 per cent of the ultimate strength, an elongation of 25 per cent in 8 in., and a reduction in area of 50 per cent at the point of fracture.

The tensile stress allowed per square in. of net cross-section in the lower flanges for dead and live load respectively is 18,000 and 9,000 for girders, 8,000 and 8,000 for floorbeams and riveted stringers, and 9,000 and 9,000 in the extreme fibers for I-beams. Shop rivets are proportioned for 15,000 lb. bearing, 7,500 lb. shear and 18,000 lb. bending. Countersunk rivets have 12,000 lb. bearing and 6,000 lb. shear. Field rivets have 12,000 lb. bearing and 6,000 lb. shear, except for beams and stringers, which have 8,000 lb. bearing and 4,000 lb. shear. In calculating the shearing and bearing stresses of web rivets, all shear on the side next to the abutment is considered transferred to the flange angles within a distance equal to the depth of the girders.

The maximum shear allowed for the webs of all girders and beams is 6,000 lb. per square in. Where there are no vertical web stiffeners, one-eighth of the gross section of the web is assumed to act as a flange. In all other cases it is assumed that no portion of the web acts as a flange. When the thickness of the web is less than one-eightieth of its unsupported depth, it is stiffened by vertical angles and fillers on both sides spaced one and one-half times the depth of the girders. Where flange reinforcement plates are used, the first one is made full length and succeeding ones are also made full length in deck girders where the ties would otherwise require to be dapped more than 1 in. All field-rivet holes are reamed through cast-iron templates. No other reaming is required except to correct imperfections in punching. The net section is determined by assuming that rupture by tension may take place either through a transverse line of rivet holes or through a diagonal line where the net section does not exceed by 30 per cent the net section of the transverse line. The maximum pressure for bed plates on masonry is 300 lb. per square in. All spans have a camber equal to the deflection under maximum loads or of $\frac{1}{1200}$ of the span.

All main girder flanges are made with 6×6-in. angles, except a few 8×8 in. for the longest spans. Nearly all flange plates are 14 or 16 in. wide, and generally do not extend more than 5 in. beyond the lines of rivets connecting them with the flange angles. In long through-girder spans the top flange

cover plates may extend 6½ in. beyond their flange angles, and will then have their edges drawn together by two outside rows of rivets. Only four or five sizes of angles are used for the web stiffeners and cross-frames, and all connections are regular. All top and bottom flanges of the same girders have the same gross cross-section. Most web plates are ⅜ in. thick, and all web stiffeners are fillered out to the thickness of the flange angles. Spans on curves are built the same as those on tangents, and skew spans are made 45 degrees right or left and the bridge squared up at the abutments. The length of the span is taken on the center line of track, and is the same as for square spans. Up to 75 ft. the spans are alike at both ends, and above that length one end has a rocker with an 8-in. plate bearing, provision being made for expansion due to a temperature variation of 150 degrees. The maximum pressure for bed plates on masonry is 300 lb. per square in. All girders rest on pedestal stones set to provide for elevations due to curvature. Tracks are spaced 14 ft. apart on centers, and through girders are spaced 14 ft. apart in the clear on tangents. In deck spans where there are two lines of girders they are spaced 7 ft. ¼ in. apart on centers for 14-in. flange cover plates and 6 ft. 10¼ in. for 16-in. plates.

All drawings are made on standard 24 × 36-in. sheets, signed by James Dunn, chief engineer, A. F. Robinson, bridge engineer. A table of weights is calculated from the materials, and if the shipping weights exceed the calculated weights by more than 2½ per cent payment is not made for the excess. If the shipping weight is materially less than the calculated weight the bridge may be rejected.

Class A includes deck girders of from 26 to 105-ft. span designed especially for economy of weight, and are used where the webs can have the most advantageous depth without regard to head room and clearance. The depth of any girder in this class is made equal to the maximum shear multiplied by the rivet pitch in inches, and divided by the allowed bearing value of rivets in the web plus 7 in. This gives 47½ in. for the depth from back to back of flange angles for a 30-ft. girder. The depth increases ¾ in. per foot up to 60 ft., where the depth is 70 in. From 60 to 90 ft. it increases 1 in. per foot; 100-ft. girders are 108½ in. deep, and 105-ft. girders are 114½ in. deep.

The general features of construction in class A, as well as in the other classes, conform essentially to familiar standards and

correspond for different spans in the same classes to the examples here shown. Lateral connection plates are shop-riveted to the flanges, and the sway-brace frames are shop-riveted together and field-riveted to the web-stiffener angles. The top lateral angles have all vertical flanges turned downwards so as to clear the ties, and it is therefore necessary to cut one angle to clear the other and to splice it at intersections. Bottom lateral angles in the same panel have their vertical flanges turned

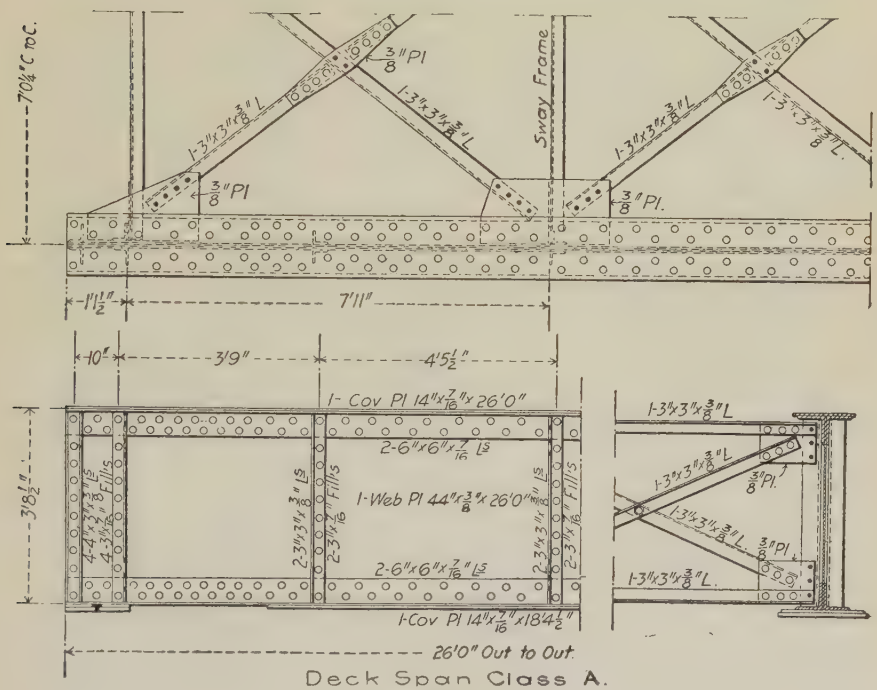


Fig. 41.

to clear each other at intersections, where they are riveted together through filler plates.

The girders are seated on shoes made with 4 × ⅝-in. Z-bars having top and bottom plates. They are anchored to the masonry and bolted to the girder flanges through slotted holes at the expansion ends, where the sole plates are beveled to allow for deflection. In long-span girders the transverse length of the bearing is increased by riveting gusset plates to the end web-stiffeners so as to produce spread footings having a length equal to the height of the girder. The rivets in the same rows

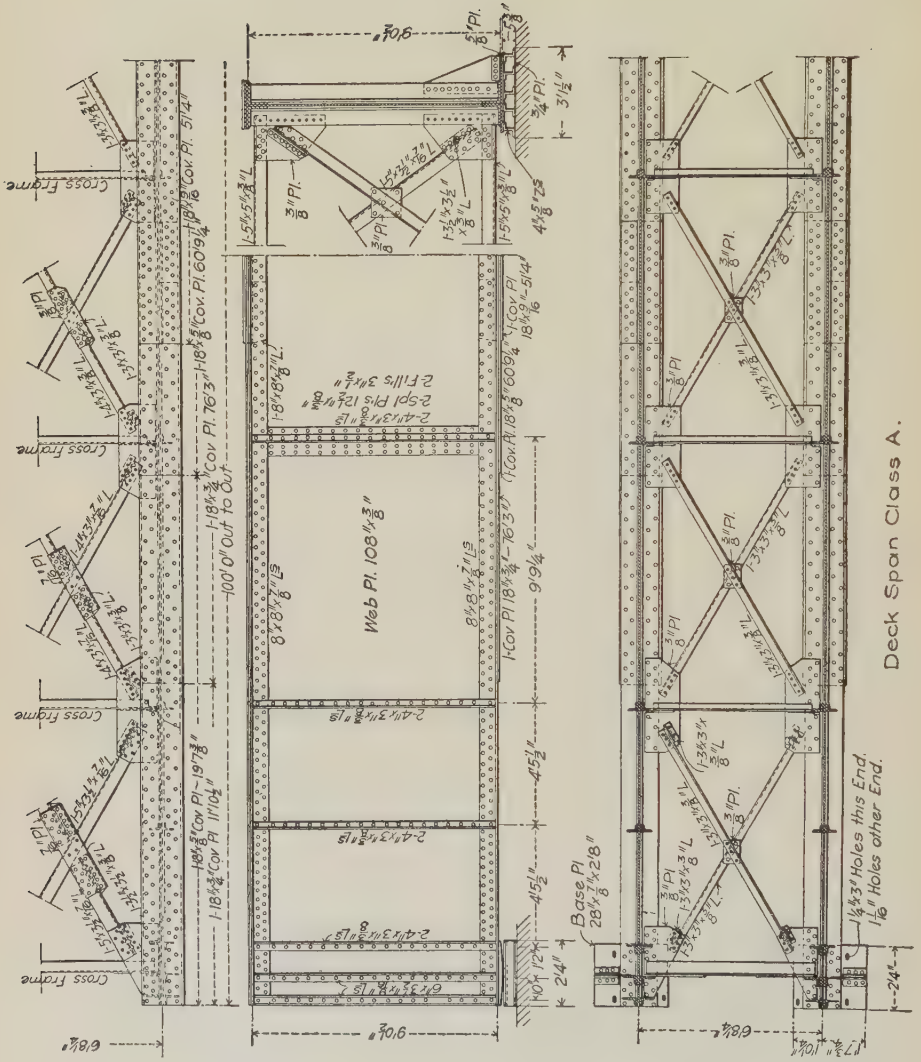
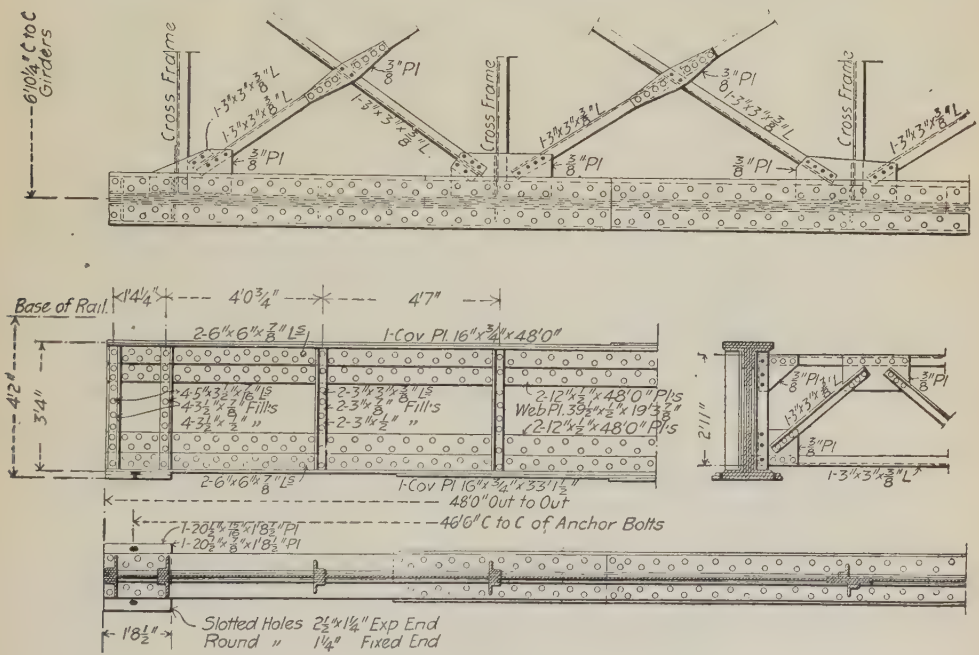

Cross Frame
Web Pl. 108 1/2 x 3/8
Base Pl 28 1/2 x 7/8 x 218"
Deck Span Class A.

Fig. 42.

in the flange angles are pitched from 2½ to 10 in. apart, and are all ⅞ in. in diameter. The top lateral bracing is proportioned for stationary and moving loads of 200 lb. and 300 lb. per lin. ft. respectively. The bottom lateral bracing is proportioned for a stationary load of 200 lb. per lin. ft.

For a 100-ft. span girder the maximum flange stresses are 130,200 lb. from dead load and 344,700 lb. from live load. The maximum shears are 47,700 lb. from dead load and 152,100 lb. from live load. The 26-ft. deck span corresponds closely to the 100-ft. span, and has maximum dead and live load flange stresses of 10,500 lb. and of 100,000 lb. with end shears of 5,940 lb. and 65,700 lb. The estimated weight of the steel is 485 lb. per cubic ft., and the floor was assumed at 1,400 lb. Wind load is calculated at 300 lb. per lin. ft. moving and 200 lb. stationary, and is all provided for by the top lateral bracing, there being no lateral system for bridges of less than 48-ft. span.



Deck Span Class B.

Fig. 43.

Class B includes deck girders of from 30 to 85-ft. span, which, on account of head room or grade, are made as shallow as possible without regard to the weight of materials. Up to 42 ft.

each track has four lines of girders and has sway-bracing, but no laterals. Above 42 ft. there are only two lines of girders, and they have top and bottom lateral systems and sway-bracing. The girders for the short spans are shop-riveted together in pairs before shipping, and are so spaced that each girder receives one quarter of the track load. The depths are usually one-

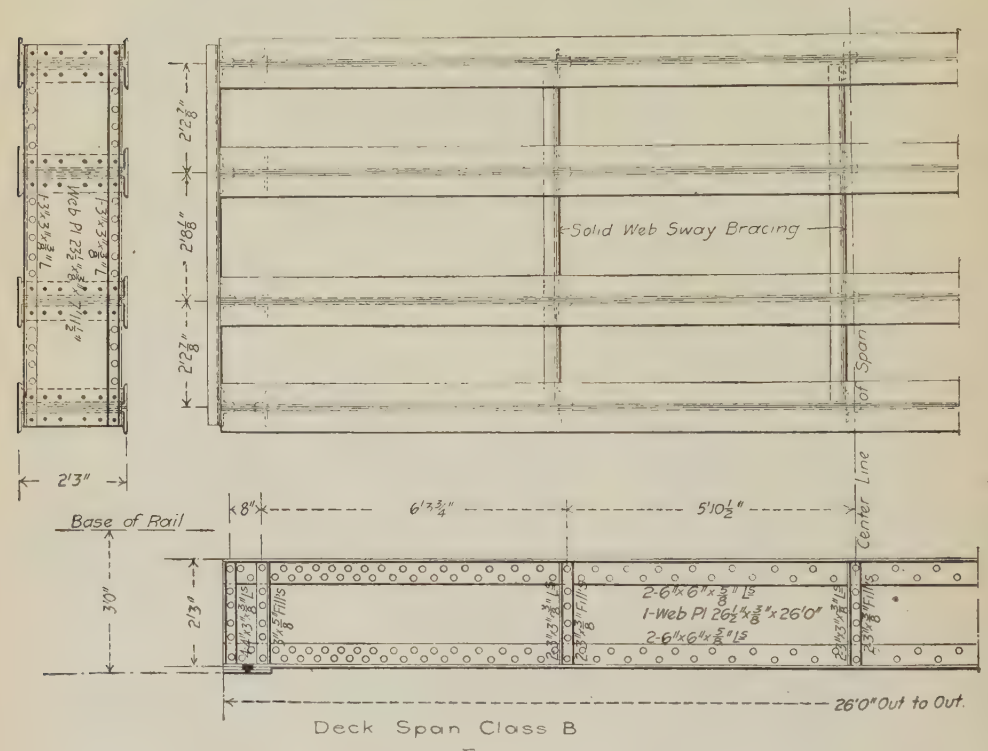


Fig. 44.

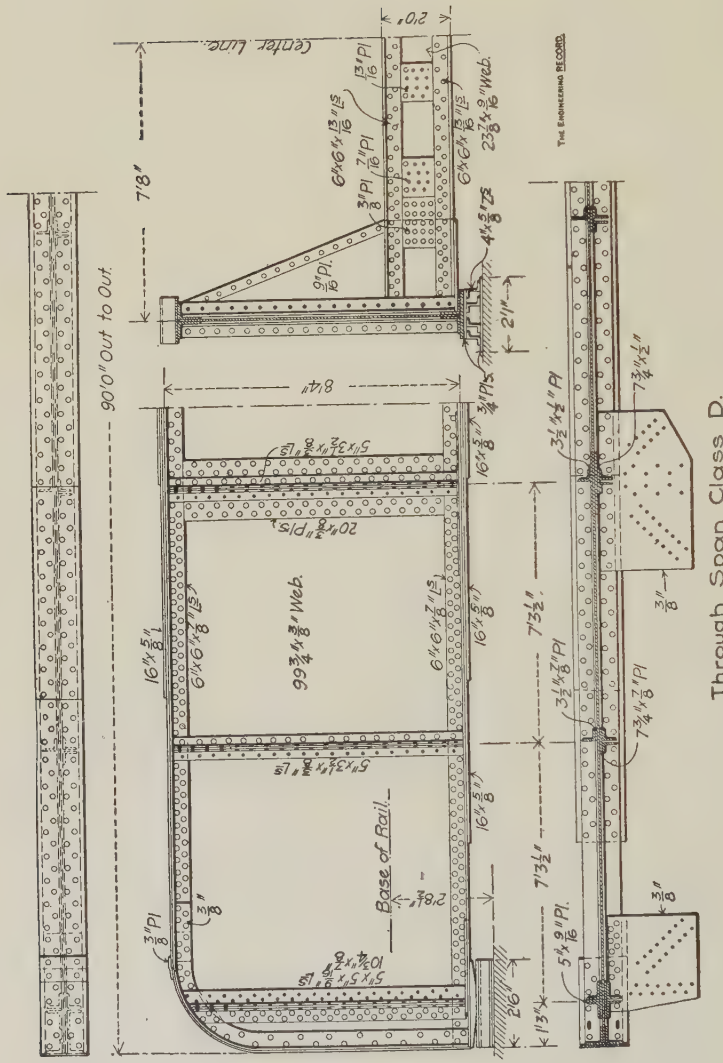
twelfth to one-fourteenth of the span, and the webs are ½ in. thick for the long spans. For spans above 40 ft. long the flanges are reinforced by 12-in. vertical side plates, as shown in the drawings of the 48-ft. span. In the 70-ft. span the maximum dead and live load flange stresses are 95,000 lb. and 359,000 lb. respectively, and the shears are 26,200 lb. and 113,200 lb. The metal weight is 1,075 lb. per lin. ft. In the 48-ft. span the maximum dead and live load flange stresses are 61,000 lb. and 304,400 lb., and the shears are 15,700 lb. and 89,200 lb. The weight of steel per linear foot is 885 lb. The short spans are sway-braced, as shown, by solid diaphragms of ⅜-in. plates

riveted to the web stiffeners and having single 3 × 3-in. top and bottom flange angles. For the 26-ft. span the maximum flange stresses for dead and live loads are 10,600 lb. and 90,000 lb., end shears 3,300 lb. and 32,800 lb. The weight of steel is 595 lb. per lin. ft.

Class D girders are used for through spans of from 26 to 106 ft., and are designed to secure economical weights in the main girders, but have shallow floors and short panels of stringers so as to reduce the head room without relation to the economical weight of floor system. The depths of the girders vary from 4 ft. 1 in. for a 32-ft. span to 8 ft. 4 in. for a 90-ft. span, and the stringers are from about 5 to 7 ft. in length. The end sections of the floorbeam webs are prolonged above the top flanges so as to form gusset plates reaching to the top flanges of the main girders and bracing them very firmly. In the short spans these gussets are proportionately very wide, and their inclined edges are stiffened by pairs of flat bars. In the 32-ft. spans the floorbeams are only 21 in. deep, and the stringers are 15-in. I-beams. The floorbeams are not more than 2 ft. deep in any span, and all stringers are made with I-beams usually of 60 lb. weight. The shorter spans have square-ended girders, but those for the longer spans are rounded on the upper corners, as shown in the elevation.

In the 90-ft. span the maximum live and dead load flange stresses are 311,400 lb. and 135,300 lb., respectively, and the shears are 118,000 lb. and 42,500 lb. The abutment reaction for each girder is 189,300 lb., and the effective depth is $8\frac{1}{3}$ ft., the steel weight is 1,800 lb. per lin. ft., and the vertical web-stiffener angles are ground at the ends to fit the top and bottom flanges. In the 32-ft. spans the maximum dead and live load flange stresses are 21,750 lb. and 11,400 lb., the shears are 8,340 lb. and 42,740 lb., and the abutment reaction is 83,760 lb. Wind load is assumed at 500 lb. moving and 500 lb. static per lin. ft., and the steel weight at 1,175 lb. per lin. ft. The floorbeams have a maximum flange stress of 171,050 lb. and shear of 53,780 lb., and weigh 3,000 lb. each. The stringers have a maximum moment of 437,760 in.-lbs. and shear of 820 dead and 18,400 lb. live.

Class C girders are used for bridges of from 60 to 106-ft. spans, and are designed with the same depth of web as class D, but have deep floorbeams and long stringers so as to give the most economical floor weights. The details correspond very



Through Span Class D.

Fig. 45.

closely to those shown for the D girders. The floorbeams are uniformly 42 in. deep, and are made with $\frac{3}{8}$-in. webs and 6 × 6-in. flange angles without reinforcement plates, and the stringers are four lines of 20-in. I-beams from 13 to 15 ft. in length. The girder stresses correspond closely to those of class D, and the total steel weight per lin. ft. is about 1,800 lb. for a 90-ft. span, 1,500 lb. for a 75-ft. span, 1,440 lb. for a 70-ft. span, and 1,400 lb. for a 60-ft.

Spans of from 8 ft. to 34 ft. are made with I-beam girders of which there are from two to five under each rail. They are connected together by transverse vertical diaphragm plates, and are seated on bent plates which extend beyond the outer flanges and receive inclined channel iron braces, bent and riveted across the top flanges. The bed plates are seated on timber cushions on the abutments.

For spans up to 9 ft. in the clear T-rails are generally used, either with or without ballast floor.

The Chicago, Burlington & Quincy Railroad.

Recent standards of design for through plate-girder spans are illustrated by the accompanying details of an 86-ft. single-track span which was erected in 1903 on the main line divisions for the Cripple Creek and for two Rock Creek bridges and at three crossings on other branches of the system. The girders are 8 ft. deep back to back of angles, 86 ft. long over all, and 15 ft. 8 in. apart on centers. The depth of floor from top of masonry to base of rail is 5 ft. $4\frac{3}{4}$ in., allowing $\frac{9}{16}$ in. for thickness of rust joint under bolster.

The girders, 84 ft. long on centers of bearings, are each proportioned for a static load of 8.4 tons per panel and for total live and dead load moments of 1,407.8 tons, 1,612.55 tons, 2,439 tons, and 2,726 tons for the first, second, third, and fourth floorbeams. These produce a maximum flange stress of 286.6 tons; the allowed unit stress is 4.72 tons, and the actual unit stress is 4.75 tons.

The web plate is made in five sections, spliced with four vertical rows of rivets through pairs of $13\frac{1}{2} \times 3\frac{3}{8}$-in. cover plates, and is stiffened by pairs of 4 × 5-in. and 4 × 6-in. vertical angles from 42 to 36 in. apart except over the end bearings, where there are four pairs in a length of 3 ft. Each flange consists of two 8 × 8 × $\frac{7}{8}$-in. angles 62 ft. 2 in. long, and two

angles 23 ft. 10 in. long, with their joints staggered and each spliced with two $\frac{7}{8}$-in. plates 6 ft. 8 in. long, as shown in the detail, one full length $30 \times \frac{11}{16}$-in., one $20 \times \frac{11}{16}$-in. plate 60 ft. 11 in. long, and one $20 \times \frac{11}{16}$-in. plate 44 ft. 11 in. long. The upper corners of the girders are square, and all vertical angles are crimped. The floorbeams 3 ft. deep have their webs made in three pieces, the end sections extending as gusset plate stiffeners to the top flanges of the girders, without stiffening angles on their inclined edges. They are field-riveted to the regular web-stiffener angles of the main girders, and to other angles shipped loose and field-riveted symmetrically with them.

The lower flanges of the floorbeams just clear those of the girders, and have transverse plates riveted to them which are bored for the $1\frac{7}{8}$-in. pins engaging the clovises of the 1-in. square lateral diagonal rods. The connection pin plates are fillered out from the floorbeam flanges so as to be flush with the girder-flange-cover plates across which they are field-riveted. The intermediate floorbeams are made with a $\frac{9}{16}$-in. web, and in each flange a pair of $8 \times 8 \times \frac{1}{2}$-in. angles and two 14-in. cover plates. The first cover plate on the lower flange is $\frac{7}{16}$-in. thick, and reaches to the lateral connection plates; the first one on the top flange is $\frac{7}{16}$-in. thick and 11 ft. 10 in. long, reaching beyond the gusset plates so that it is split about 10 in. at each end to clear them; both outer cover plates are $\frac{3}{8}$ in. thick and 9 ft. 4 in. long. The end floorbeams are similar, but have 6×4-in. flange angles; and are somewhat lighter.

The stringers have $28 \times \frac{9}{16}$-in. webs and pairs of $6 \times 4 \times \frac{1}{2}$-in. flange angles with single full-length $10\frac{1}{2} \times \frac{1}{2}$-in. cover plates. The top flange angles are cut 6 in. short at each end, and the vertical legs of the bottom flange angles are notched at each end to clear the 6×6-in. vertical end connection angles which have their outstanding flanges milled and are connected to the floorbeams with eleven rivets each. The end floorbeams have solid-web brackets 2 ft. wide riveted to them on the shore side in the planes of the stringers to carry the ties over the bridge seats.

Cast-iron shoes are secured to the bottom flanges of the main girders, 24 ft. apart on centers, by eight $1\frac{1}{8}$-in. tap bolts in each. They have continuous semicircular bearings for 4-in. pins with a projecting $2 \times \frac{1}{4}$-in. guide rib in the middle. The pins engage pedestals which are substantially duplicates of the shoes, except that they have $2\frac{1}{4} \times \frac{1}{2}$-in. center longitudinal guide ribs

projecting from the base plates to engage slots in the tops of the extended cast-iron webbed bolsters 12 in. high to which they are secured by two $1\frac{1}{4}$-in. bolts in slotted holes. The anchor bolt holes through the bolsters are slotted to allow for adjustment in setting. Bearings at both ends of the span are alike, so as to permit temperature movements at both ends of the span.

A 105-ft. deck span dated February, 1902, is one of the longest plate-girder spans on this road, and illustrates the construction then adopted but which has since been changed in some particulars in accordance with the development of the present specifications and latest standards. The girders are 10 ft. deep back to back of angles and 8 ft. apart on centers. The web is $\frac{5}{8}$-in. thick throughout, and is made in lengths of $10\frac{1}{4}$ to 12 ft. The maximum spacing of stiffeners is only 6 ft., and there are only two pairs of them over each end bearing. The maximum flange section consists of two $8 \times 8 \times \frac{7}{8}$-in. angles, spliced 39 ft. from the ends of the girder, and three $19 \times \frac{1}{16}$-in. and one $19 \times \frac{3}{4}$-in. cover plate. The flange angles are spliced with $8 \times 8 \times \frac{5}{8}$-in. cover angles 8 ft. long, and the shoes are bolted to planed $19 \times 1 \times 24$-in. sole plates with their edges milled to bear against the ends of the first cover plate and countersunk riveted to the bottom flanges of the girders. Otherwise, except for the omission of the floorbeam connections, the girders correspond to those described for the 86-ft. span.

The span is divided into twelve panels by vertical transverse cross-frames made with single top and bottom horizontal angles and X-brace angles riveted to the vertical web-stiffener angles. Each panel, between cross-frames, is braced in the plane of the lower flanges by a single diagonal lateral angle varying from 6×4 in. at the ends to 3×3 in. in the center of the span. There are no lateral diagonals in the plane of the top flanges. The shoes, pedestals, and bolsters are similar to those for the 86-ft. span, but differ materially in detail. The upper surfaces of the shoes are planed, and have short flanges engaging the edges of the sole plates on the girders. Their 5-in. pins engage pedestals which are unsymmetrical about the center line of the bearings. The longitudinal guide rib in the base of the pedestal is only $\frac{1}{2}$ in. high at the fixed end where it engages the bolster casting directly.

At the sliding end this rib is $1\frac{1}{2}$ in. high, and engages the planed edges of the phosphor-bronze bearing plates 1 in. thick,

which are tap-bolted to the top of the bolster, and with the pedestal base are coated with graphite and tallow. Each bolster is anchored to the masonry with four $1\frac{1}{4}$-in. rag bolts 11 in. long, which pass through round holes in the base of the expansion end bolster and through slotted holes in at the fixed bolsters. The bolsters are set on $\frac{5}{8}$-in. rust joints, and the base of rail is 13 ft. $7\frac{3}{4}$ in. above the top of masonry.

The girders were each proportioned for a dead load of 0.52 ton per lin. ft.; the maximum flange stress is 324.94 tons, allowed unit stress 4.64 tons; required net flange sections 70 sq. in., actual net flange section 70.77 sq. in. Four spans were built in 1902 across the Nishnabotna River, and three spans were built in 1903 across the Cuivre River. All designs were made and construction supervised by the engineering department of the railroad; Mr. W. L. Breckenridge, chief engineer, and Mr. C. H. Cartlidge, bridge engineer.

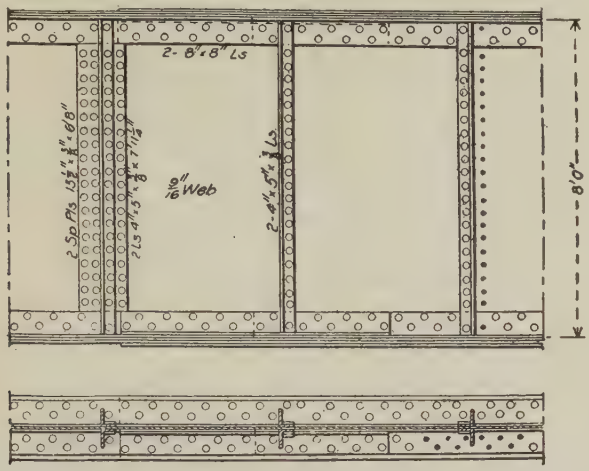
Latest Structures.

Under date of May 9, 1907, Mr. Cartlidge writes: "I regret that the above examples are now so badly out of date that I feel we are hardly being done justice to. . . . I have instructed my office to send you blueprints of some of our standard constructions. We do not consider any bridge which does not have a ballasted floor as being of standard construction, and you will note that both through and deck girders are designed accordingly, although either plan may be used with wooden floors."

The bridges referred to are a 75-ft. deck and a 75-ft. through span, and a 105-ft. deck span. The 75-ft. through span has two girders 15 ft. 6 in. apart on centers and 8 ft. $4\frac{1}{4}$ in. deep back to back of flange angles. The web is made with three nearly equal lengths of $100 \times \frac{9}{16}$ in. plates shop-spliced with four vertical rows of rivets at each joint through a pair of $12 \times \frac{3}{8}$-in. vertical cover plates without stiffener angles at each joint, and by two $7\frac{1}{4} \times \frac{1}{2} \times$ 6-ft. 4-in. plates over the vertical legs of the flange angles. It is divided into panels about 9 ft. long by pairs of $4 \times 4 \times \frac{1}{2}$-in. vertical stiffener angles besides intermediate pairs of crimped angles in the middle of the end panels, and two pairs of $6 \times$ 6-in. angles and $14 \times \frac{3}{4}$-in. cover plates over the end bearings.

The webs are only 5 ft. 4 in. deep at the ends, sloping up to join the full depth sections at the first panel point with a short tangent curve of 6 ft. radius. Two horizontal lines of angles

18 in. apart in the clear are field-riveted to the inner face of the web with their vertical legs turned apart to form connections for ballasted floor construction, the top of the $4 \times 4 \times \frac{3}{8}$-in. lower angles being flush with the top flanges of the stringers. The top and bottom flanges are alike and are each made with two $8 \times 8 \times \frac{3}{4}$-in. angles, one $18 \times \frac{5}{8}$-in., and one $18 \times \frac{1}{2}$-in.



Girder Splice.

Fig. 49.

cover plate, all full length pieces except the last, which is 35 feet long.

The floorbeams, $26\frac{1}{2}$ in. deep, are made with two $6 \times 6 \times \frac{1}{2}$-in. angles and two $14 \times \frac{3}{8}$-in. cover plates in each flange. The end sections of the webs extend beyond the top flanges to make gusset plates, nearly the full depth of the main girders, with single crimped vertical angles field-riveted to the web stiffeners and to the webs through both flanges. There are seven lines of 15-in. 42-lb. I-beam stringers about 9 ft. long web-connected to the floorbeams by 14 rivets at each end. There are also two outside lines of stringers of the same size connected to the floorbeams 26 in. from the girder webs by 8 rivets in each end.

Pairs of $\frac{3}{8}$-in. horizontal connection plates field-riveted to the lower flanges of the floorbeams and girders on both sides of the webs of the former receive the field-riveted connections of the $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$-in. X-brace lateral angles in the panels be-

tween floorbeams. One angle in each panel is continuous, and the other is cut to clear, and spliced over it with a flange plate.

The bottom flange cover plates are cut 15 in. short at each end to clear a 1-in. planed sole plate with two 6 × 1½-in. holes for anchor bolts. The girders are seated on the planed tops of cast iron pedestals 7⅜ in. high with 20 × 36-in. bases, and one center longitudinal and four transverse webs. There are guide ribs on each longitudinal edge of the upper surface, and the base plate has four slotted holes for stone bolts 11 in. long.

The main girders have a maximum moment of 28,817 tons with a net sectional area of 44.88 square in. for an allowed unit stress of 6.65 tons. The maximum shear in the end panel is 113.32 tons. The maximum moment of the interior floorbeam is 2,543 in.-tons and the net flange area 18.5 square in. For the stringers the moment is 641 in.-tons.

In the 75-ft. deck spans the girders correspond to those in the through spans except that their webs are of the same depth throughout; that the flange angles are reinforced with two 18 × ½-in. and one 18 × $\frac{9}{16}$-in. cover plates; and that the web stiffeners, 6 ft. apart, are pairs of crimped 5 × 3½ × $\frac{7}{16}$-in. angles without floorbeam connections. Connection plates for the top lateral zigzag angles and the sway-brace frames are field-riveted between the top flanges and short horizontal connection angles shop-riveted to the girder webs. The girders are spaced 7 ft. apart on centers and are seated on unsymmetrical cast-iron pedestals similar to those of the through span but having 24 × 36-in. bases.

The maximum moment of 28,970 in.-tons requires a net flange area of 44.88 square in. The allowed flange stress with concrete deck is 6.49 tons. The weight of one girder is 43,900 lb. The total weight of one span is 95,800 lb. The corresponding weights of the girder and the span for the through construction are 40,860 and 144,400 lb.

The new 105-ft. deck plate girder span corresponds essentially with those above described for the Nishnabotna and Cuivre rivers, the principal difference being in the allowance of 1.2 tons per linear ft. additional dead load due to concrete deck which raises the unit flange stress from 4.86 tons for ordinary floor to 5.5 tons; and in the abutment bearings, which are made alike at both ends with the pedestals sliding directly on the bolsters without the intervention of the phosphor bronze

plates previously used. One span over Pope Creek on the Galesburg Division weighs 208,000 lb.

The provision for concrete floors in deck girder spans includes the lowering of the plane of the top lateral system and the top angles of the sway-brace frames to the lower edge of the top flange angles. This permits the forms for the deck concrete to be set with greater ease and to be supported on the transverse frames and lateral angles. The outstanding flanges of the vertical web-stiffener angles in deck girders are punched for connection bolts for the concrete forms for the projecting portions of the deck which are cantilevered and knee-braced from the top flanges of the girders.

The most important differences between the present and recent standards of this railroads are that now girder spans of 80 ft. and over have roller expansion bearings; that web stiffeners intermediate between the floorbeams are omitted in through girders; and that both through and deck spans are designed for reinforced concrete floors.

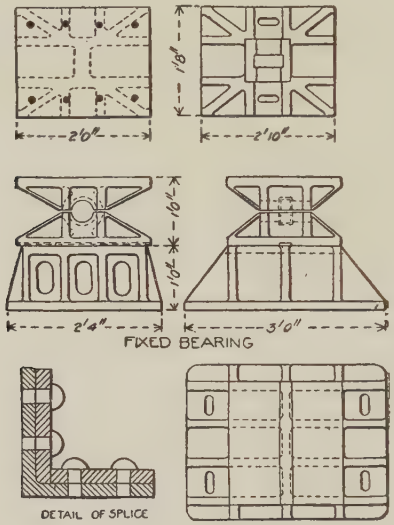
Baltimore & Ohio Railroad.

A complete series of standard designs for single-track plate girder bridges has been prepared by Mr. J. E. Greiner, Engineer of Bridges, and adopted by the Baltimore & Ohio Railroad. They are made in accordance with the regular Baltimore & Ohio Railroad Bridge Specifications of 1901, based on a unit stress of 16,000 lb. tension for open-hearth steel and all rivets $\frac{7}{8}$ in. in diameter. The essential data have been computed and arranged in a very compact table which gives all principal materials and dimensions for spans varying by 5-ft. increments from 30 to 100 ft. This with one sheet of general diagrams and one sheet of standard details for end bearings, lateral bracing and sway frames, affords complete contract data from which the shop drawings can be prepared with uniformity for any deck span within the limits for which they are generally used on this road.

It is assumed that the flange angles and plates are single unspliced pieces and that the net area of the web in the flange section allows for the unspliced portion of the web. All spans are made with transverse sway bracing and with zigzag top flange lateral angles; the longest spans only, have bottom flange lateral bracing also. Up to and including 75-ft. spans all girders

are $6\frac{1}{2}$ ft. apart on centers; above that length they are 8 ft. apart, and the ties are changed from 8×8 in. 9 ft. long to 8×10 in. $10\frac{1}{2}$ ft. long.

Spans less than 80 ft. long are like the 50-ft. span, and are seated at both ends on cast-iron pedestals to which they are bolted through round and slotted flange holes at the fixed and expansion ends respectively. The pedestals are planed only if the bases vary more than $\frac{1}{16}$ in. from true planes.



Fixed Bearing.

Fig. 50.

They are secured to the masonry by two $1\frac{1}{4}$-in. rag bolts. The sway frames are field-riveted to the vertical web-stiffener angles, and the lateral angles have horizontal connection plates field-riveted to them and to the under sides of the top flanges of the girders. Where there are no cover plates on the flanges there are $\frac{5}{8}$-in. filler plates between the connection plates and the flange angles to offset the lateral angles clear of the dapped ties.

Spans above 75 ft. long correspond to the 80-ft. span, and have pin bearings with cast-steel shoes and pedestals and 4-in. pins. The expansion pedestals are seated on roller nests and riveted bed plates, and the fixed pedestals are made 6 in. higher than the expansion pedestals to compensate for the

TABLE 30.

Assumed Live Loading	177½-ton Engine including Tender (Cooper's E-50) with Impact.				
Length out to out of girders..	30 — 0	35 — 0	40 — 0	45 — 0	50 — 0
Length c. to c. bearings	28 — 0	33 — 0	38 — 0	43 — 0	48 — 0
Depth back to back of L's ..	4 — 0	4 — 6	5 — 0	5 — 6	6 — 0
Total moment ft.-lbs., one rail	310660	1196900	1500800	1854800	2215700
Flange stress.....	247600	265900	300000	337300	370000
Flange area req'd.....	15.5	16.6	18.8	21.1	23.1
Flange Section	⅜ web = 2.2 2 L's 6 × 6 × ⅝ = 13.0	⅜ web = 2 5 2 L's 6 × 6 × ½ = 9.5 I-Pl. 16 × ⅜ × 35 = 5.3	web = 1.0 2 L's 6 × 6 × ⅝ = 11.7 I-Pl. 16 × 7/16 × 40 = 6.1	web = 1.2 2 L's 6 × 6 × ⅝ = 11.7 I-Pl. 16 × 9/16 × 45 = 7.9	web = 1.5 2 L's 6 × 6 × ⅝ = 11.7 I-Pl. 16 × 11/16 × 50 = 5.3 I-Pl. 16 × ⅝ × 26 = 5.3
Total net area sq. inches	15.2	17.3	18.8	20.8	23.8
Total shear.....	151000	167400	182700	197300	211000
Web area req'd.....	15.1	16.7	18.3	19.7	21.1
Web section.....	47½ × ⅜	53½ × ⅜	59½ × ⅜	65½ × ⅜	71½ × ⅜
End pitch through flange	2¼	2½	2¼	2½	2¼
End stiffeners (with fillers)...	5 × 3½ ×	5 × 3½ ×	5 × 3½ ×	5 × 3½ ×	5 × 3½ ×
Interm. stiffeners	3½ × 3½ ×	3½ × 3½ ×	3½ × 3½ ×	3½ × 3½ ×	3½ × 3½ ×
End cross frames	6 × 4 ×	6 × 4 ×	6 × 4 ×	6 × 4 ×	6 × 4 ×
Interm. cross frames.....	3½ × 3½ ×	3½ × 3½ ×	3½ × 3½ ×	3½ × 3½ ×	3½ × 3½ ×
Laterals { top.....	6 × 4 ×	6 × 4 ×	6 × 4 ×	6 × 4 ×	6 × 4 ×
Laterals { bottom					
Size of bed plates.....	24 × 24	24 × 24	24 × 24	24 × 24	24 × 24
Base of rail to masonry	5 — 8¼	6 — 3	6 — 9	7 — 3¼	7 — 9
Ties	8 × 8 × 9 — 0	8 × 8 × 9 — 0	8 × 8 × 9 — 0	8 × 8 × 9 — 0	8 × 8 × 9 — 0
Approximate total weight....	16300	20000	25000	31500	35500

TABLE 30 (*Continued*).

Assumed Live Loading.	177½-ton Engine including Tender (Cooper's E-50) with Impact.				
Length out to out of girders..	55 — 0	60 — 0	65 — 0	70 — 0	75 — 0
Length c. to c. bearings.....	53 — 0	58 — 0	63 — 0	68 — 0	73 — 0
Depth back to back of L's...	6 — 6	7 — 0	7 — 0	7 — 6	7 — 6
Total moment ft.-lbs., one rail	2611600	3046300	3521300	4033200	4555100
Flange stress.....	401800	435200	503000	537700	607300
Flange area req'd.....	25.1	27.2	31.4	33.6	37.9
Flange section	web = 1.7 2 L's 6 × 6 × ⅝ = 11.7 I-Pl. 16 × 7/16 × 55 = 6.1 I-Pl. 16 × 7/16 × 29 = 6.1	web = 2.0 2 L's 6 × 6 × ⅝ = 11.7 I-Pl. 16 × ½ × 60 = 7.0 I-Pl. 16 × ½ × 33 = 7.0	web = 2.0 2 L's 6 × 6 × ⅝ = 11.7 I-Pl 16 × 9/16 × 65 = 8.8 I-Pl. 16 × 9/16 × 38 = 8.8	web = 2.3 2 L's 6 × 6 × ⅝ = 11.7 I-Pl. 16 × ⅝ × 70 = 7.0 I-Pl. 16 × ⅝ × 45 = 7.0 I-Pl. 16 × ⅝ × 33 = 7.0	web = 2.3 2 L's 6 × 6 × ⅝ = 11.7 I-Pl. 16 × 11/16 × 75 = 7.9 I-Pl 16 × 11/16 × 52 = 7.9 I-Pl. 16 × 11/16 × 38 = 7.9
Total net area sq. inches.....	25.6	27.7	31.3	35.0	37.7
Total shear.....	223800	237300	251000	267400	283500
Web area req'd.....	22.4	23.7	25.1	26.7	28.4
Web section.....	77½ × 5/16	83½ × 5/16	83½ × 5/16	89½ × ⅜	89½ × ⅜
End pitch through flange	2¼	2¼	2¼	2¼	2¼
End stiffeners (with fillers) ...	5 × 3½ × 5/16	5 × 3½ × 5/16	5 × 3½ × 5/16	5 × 3½ × ⅜	5 × 3½ × ⅜
Interm. stiffeners	3½ × 3½ × ½	5 × 3½ × ½	5 × 3½ × ½	5 × 3½ × ½	5 × 3½ × ½
End cross frames	6 × 4 × ½	6 × 4 × ½	6 × 4 × ½	6 × 4 × ½	6 × 4 × ½
Interm. cross frames.....	3½ × 3½ × ½	3½ × 3½ × ½	3½ × 3½ × ½	3½ × 3½ × ½	3½ × 3½ × ½
Laterals { top	6 × 4 × ½	6 × 4 × ½	6 × 4 × ½	6 × 4 × ½	6 × 4 × ½
Laterals { bottom					
Size of bed plates.....	24 × 24	24 × 30	24 × 30	24 × 30	24 × 30
Base of rail to masonry.....	8 — 3	8 — 9¼	8 — 9½	9 — 3¼	9 — 3¼
Ties	8 × 8 × 9 — 0	8 × 8 × 9 — 0	8 × 8 × 9 — 0	8 × 8 × 9 — 0	8 × 8 × 9 — 0
Approximate total weight	40500	48500	54000	65000	70000

TABLE 30 (*Continued*).

Assumed Live Loading.	177½-ton Engine including Tender (Cooper's E-50) with Impact.				
Length out to out of girders..	80 — 0	85 — 0	90 — 0	95 — 0	100 — 0
Length c. to c. bearings.....	77 — 0	82 — 0	87 — 0	92 — 0	97 — 0
Depth back to back of L's ...	8 — 0	8 — 6	9 — 0	9 — 0	9 — 0
Total moment ft.-lbs., one rail	5011000	5599000	6223000	6880000	7557000
Flange stress	626400	658700	691400	764500	839500
Flange area reg'd.....	39.2	41.1	43.2	47.8	52.5
Flange section.....	web = 2.0 2 L's 8 × 8 × $\frac{5}{8}$ = 16.7 I-Pl. 18 × $\frac{6}{8}$ × 80 = 10.0 I-Pl. 18 × $\frac{6}{8}$ × 44 = 10.1	web = 2.4 2 L's 8 × 8 × $\frac{5}{8}$ = 16.7 I-Pl. 18 × $\frac{5}{8}$ × 85 = 10.0 I-Pl. 18 × $\frac{6}{8}$ × 46 = 10.0	web = 2.8 2 L s 8 × 8 × $\frac{5}{8}$ = 16.7 I-Pl. 18 × $\frac{1}{2}$ × 90 = 8.0 I-Pl. 18 × $\frac{1}{2}$ × 58 = 8.0 I-Pl. 18 × $\frac{1}{2}$ × 42 = 8.0	web = 2.8 2 L's 8 × 8 × $\frac{3}{4}$ = 19.9 I-Pl. 18 × $\frac{9}{16}$ × 95 = 9.0 I-Pl. 18 × $\frac{9}{16}$ × 60 = 9.0 I-Pl. 18 × $\frac{9}{16}$ × 44 = 9.0	web = 2.8 2 L's 8 × 8 × $\frac{3}{4}$ = 19.9 I-Pl. 18 × $\frac{6}{8}$ × 100 = 10.0 I-Pl. 18 × $\frac{6}{8}$ × 65 = 10.0 I-Pl. 18 × $\frac{6}{8}$ × 46 = 10.0
Total net area sq. inches....	38.8	39.1	43.5	48.7	52.7
Total shear..	299300	313000	328800	344000	357700
Web area reg'd.....	29.9	31.3	32.9	34.4	35.8
Web section.....	9½ × $\frac{7}{16}$	10½ × $\frac{7}{16}$	10½ × $\frac{7}{16}$	10½ × $\frac{7}{16}$	10½ × $\frac{7}{16}$
End pitch through flange	2¼	2¼	2¼	2¼	2¼
End stiffeners (with fillers)...	6 × 4 × $\frac{1}{2}$	6 × 4 × $\frac{1}{2}$	6 × 4 × $\frac{1}{2}$	6 × 4 × $\frac{1}{2}$	6 × 4 × $\frac{1}{2}$
Interm. stiffeners.....	6 × 4 × $\frac{3}{8}$	6 × 4 × $\frac{3}{8}$	6 × 4 × $\frac{3}{8}$	6 × 4 × $\frac{3}{8}$	6 × 4 × $\frac{3}{8}$
End cross frames	6 × 4 × $\frac{1}{2}$	6 × 4 × $\frac{1}{2}$	6 × 4 × $\frac{1}{2}$	6 × 4 × $\frac{1}{2}$	6 × 4 × $\frac{1}{2}$
Interm. cross frames.....	6 × 4 × $\frac{3}{8}$	6 × 4 × $\frac{3}{8}$	6 × 4 × $\frac{3}{8}$	6 × 4 × $\frac{3}{8}$	6 × 4 × $\frac{3}{8}$
Laterals { top.....	6 × 4 × $\frac{1}{2}$	6 × 4 × $\frac{1}{2}$	6 × 4 × $\frac{1}{2}$	6 × 4 × $\frac{1}{2}$	6 × 4 × $\frac{1}{2}$
Laterals { bottom	6 × 4 × $\frac{3}{8}$	6 × 4 × $\frac{3}{8}$	6 × 4 × $\frac{3}{8}$	6 × 4 × $\frac{3}{8}$	6 × 4 × $\frac{3}{8}$
Size of bed plates.....	36 × 38	36 × 38	36 × 38	36 × 38	36 × 38
Base of rail to masonry	10 — 11½	11 — 5½	11 — 11¼	11 — 11¼	11 — 11½
Ties	8 × 10 × 10 — 6	8 × 10 × 10 — 6	8 × 10 × 10 — 6	8 × 10 × 10 — 6	8 × 10 × 10 — 6
Approximate total weight	95500	101500	114000	127500	135000

roller nests and allow uniform levels of the substructure masonry. The lateral and sway-bracing angles are like those for the shorter spans, except that they have more rivets in their connections and that they all are 6×4 in. All the work is notably solid and massive, designed for stiffness and durability under high train speeds and heavy loads. For short through-spans, two plate girders are used to carry the two tracks. They are made with rounded upper corners, and have their end base plates extended beyond the edges of the bottom chord angles and stiffened with transverse vertical gusset plates riveted to the end web-stiffener angles. The lower chords of the floorbeams are flush with the lower chords of the girders, and solid web kneebrace brackets are riveted to the upper sides of the floorbeams at both ends, so as to extend their connections over the whole depth of the girders. The stringers are made with web-stiffener angles, and the four lines are connected together by transverse angles riveted across their top chords. Some of the girders are 95 ft. long and weigh 36 tons each. They have webs 10 feet deep, 8×8-in. chord angles and 18-in. cover plates.

The deck spans have an independent pair of plate girders under each track. They have square corners, and are connected together by transverse struts and diagonals forming sway-brace frames at every second web stiffener. The top chords are connected by zigzag lateral diagonals connected at every web stiffener. All braces are single angles.

The Northern Pacific Railway Former Standard.

Single-track plate-girder spans of the standard of 1896 include lengths of from 36 to 100 ft. All rivets are $\frac{7}{8}$ in. in diameter in reamed holes, and all vertical web stiffeners have driving fits. All masonry plates are bedded on $\frac{1}{4}$-in. sheet lead. Short spans are made entirely of soft steel, and those of 70 ft. and over are made of soft steel except end pins, shoes, rollers, and bed plates of medium steel and bolsters of cast steel. On tangents deck girders are spaced from $6\frac{1}{2}$ to 8 ft. apart on centers according to spans, and through girders are spaced 12 ft. apart in the clear. Ties 9×9 in. and 12 ft. long are dapped on the stringers and secured to them by hook bolts and spaced 14 in. apart on centers.

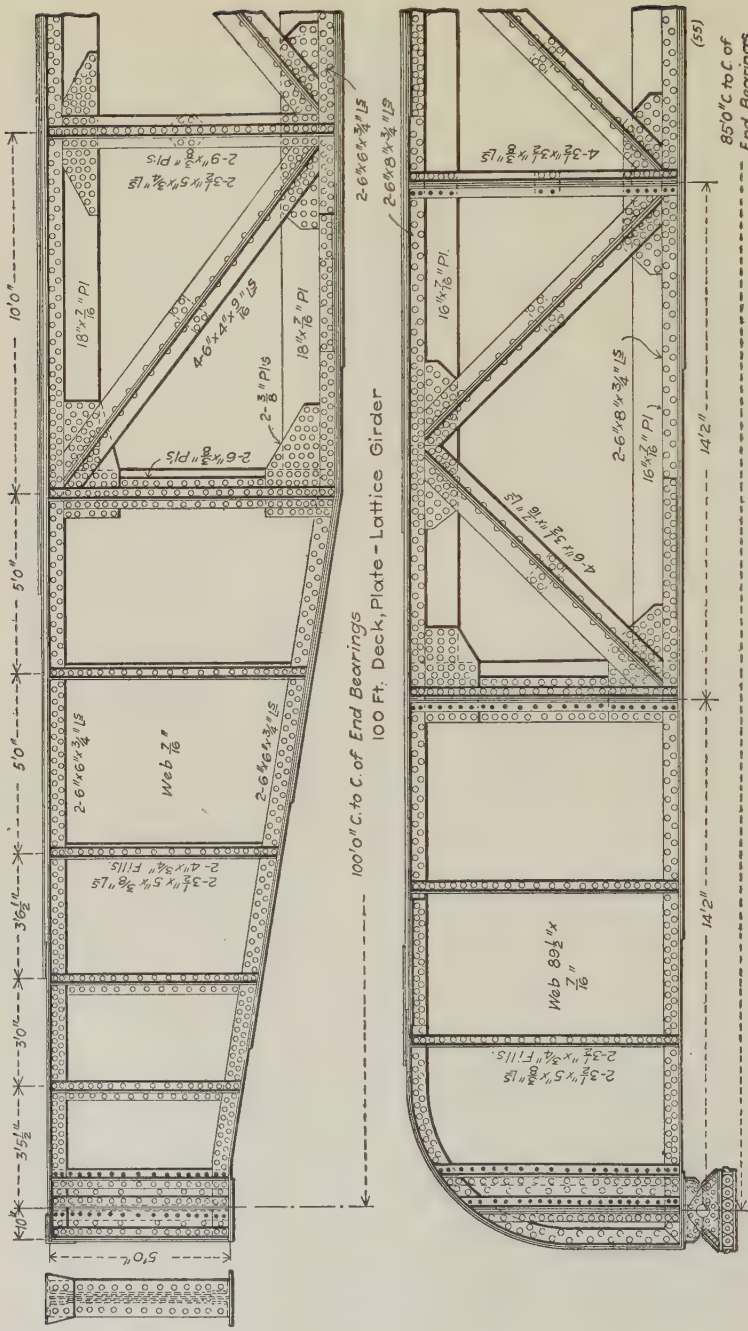
All deck spans have top and bottom lateral zigzag angles and

sway-brace frames riveted to the horizontal connection plates on the inside flange angles. Up to 50 ft. the top and bottom flanges are parallel and the ends are square. Spans of 36 ft. are 4 ft. deep. The flange angles project about $\frac{1}{4}$ in. beyond the edges of the web. Vertical web-stiffener angles are in pairs symmetrically placed on opposite sides of the web, and have fillers of the thickness of the flange angles, which at web splices are wide enough for four vertical rows of rivets.

At splices the web flanges of the stiffener angles are reversed and their outstanding angles have their backs in the same transverse plane, one on each web section. The end web-stiffener angles have flange cover plates with short ends bent at right angles over the top flanges. The 50-ft. girders are 4 ft. deep with 4 × 6-in. flange angles and 9-in. cover plates. The 3 × 3-in. web-stiffener angles are from $3\frac{1}{2}$ to 6 ft. apart. Both end bearings are simple sole plates planed and anchor-bolted through flat-planed bed plates with guide strips riveted on each side to engage the edges of the sole plate.

The 50-ft. spans are the same, except that they are $5\frac{1}{2}$ ft. deep, and have the 4-in. legs of the flange angles riveted to the webs, and the 6-in. legs riveted to 12-in. cover plates. The 70-ft. spans are $6\frac{1}{2}$ ft. deep for 46 ft. 8 in. in the center, and their bottom flanges in type II are beveled to make them only 5 ft. deep at the ends. They have 6 × 6-in. flange angles spliced with 5 × 5-in. cover angles. The cover plates are 18 in. wide next to the flange angles and 12 in. wide outside. Shoes are riveted to the ends of the bottom flanges, and have 4-in. pins engaging pedestals which at the fixed ends are seated on cellular cast bed plates and at the expansion ends are seated on nests of $3\frac{1}{2}$-in. rollers in dust-proof frames with curved side bars.

Type I is like type II, except that the web is of uniform depth throughout. In some cases there are special rocker-pin bearings for the expansion ends. The 75-ft. through plate-girder spans have their upper corners rounded and the top flange angles bent to a radius of 3 ft. and continued down vertically to the lower flanges where they are mitered together. In these spans the floorbeam webs are made in three pieces, and the short end sections project beyond the top flanges to make triangular kneebrace gusset connection plates extending to the top flanges of the main girders and field-riveted through their entire length to the vertical web-stiffener angles. The



'85 Ft. Through, Plate-Lattice Girder.

Fig. 51.

edges of the kneebraces are stiffened with pairs of flange angles, and there is an oblique angle cover plate at the intersection with the top flange of the floorbeam. The 15-ft. stringers are 23 in. deep with pairs of 6 × 3½-in. flange angles without cover plates. They have zigzag angles for sway bracing and in the planes of the top flanges.

The 100-ft. deck spans are made with several variations. All of them are 96 in. deep, and have 6 × 6-in. flange angles and 20-in. cover plates and bearings with pins, pedestals, shoes, and bolsters as already described. They are made with the bottom flange horizontal throughout, or with it beveled upwards to reduce the depth to 60 in. at one or both ends. All of these types are made either with a solid $\frac{7}{16}$-in. web throughout or with open panels braced by zigzag angles in the center. They are cambered 1 in. before erection, which is calculated to leave ¾-in. camber under dead load only and ½ in. under live load. The regular through-span 100-ft. girders correspond in general to the 100-ft. deck girders, except that the upper corners are rounded, and that instead of being shipped whole they are spliced in two places each with field-driven rivets.

Girders with open panels have web plates between the flange angles which are reinforced by side plates to distribute the stresses from the vertical and diagonal members. The diagonals are made with two pairs of angles back to back, and the verticals with a wide web plate and an angle on each side of it. The shallow floor spans correspond to those shown, except that the floorbeams are only 30 in. deep, and that for about 25 ft. at each end the top flanges are sloped down to give a depth of 6 ft. to the ends of the webs which are rounded to curves of 3-ft. radius.

The Northern Pacific Railway Revised Standard.

In a paper published in the Journal of the Western Society of Engineers, February, 1901, Mr. Ralph Modjeski, C. E., described a set of designs for standard single-track plate-girder bridges which he had made and revised for the Northern Pacific Railway. The original plans were made for a loading of two 116-ton locomotives and a train load of 3,000 lb. per lin. ft., but the weight of the rolling stock had increased so much in a few years that it was necessary to provide 146-ton locomotives and 4,000-lb. train load, and the tonnage that was involved was so great that it was found to be cheaper to make new drawings

for all spans in which some of the former details were superseded by new ones. The plate-girder spans are from 25 to 100 ft. long, varying by increments of 5 ft., and include viaducts with 30 and 40-ft. towers from 30 to 70 ft. apart, varying by 10-ft. increments. The drawings show all details and are intended for use as shop plans.

In designing the girder flanges the angles and cover plates only were considered in calculating the moment of resistance to flexure, but for beams and stringers the web was considered in the flange areas. All rivets are $\frac{7}{8}$ in. diameter of soft steel, all other material of medium steel. Rivet holes punched $\frac{1}{16}$ in. and reamed to $\frac{5}{16}$ in. Maximum stresses per sq. in. for rolling

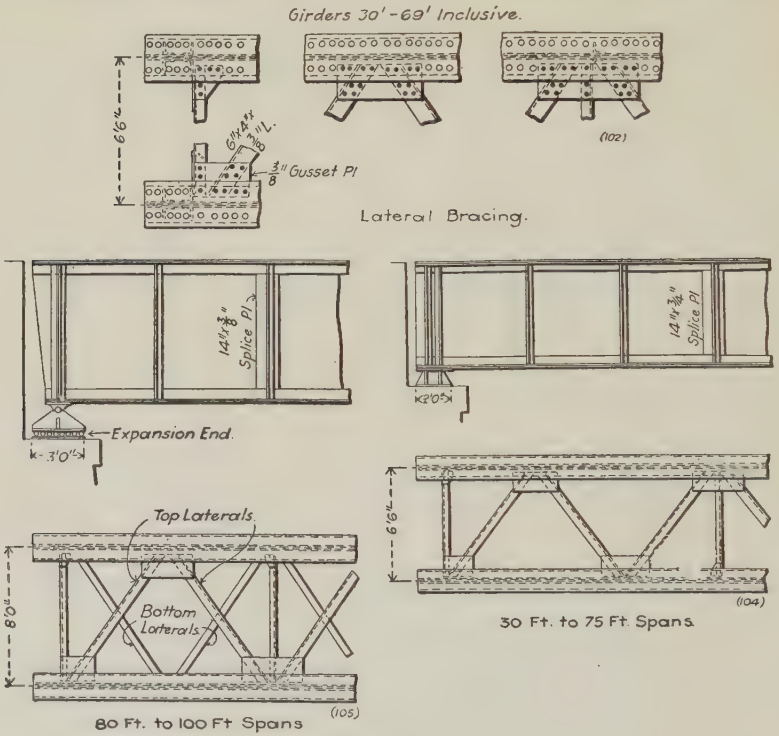


Fig. 52.

and dead loads respectively, 10,500 lb. and 18,000 lb. respectively. An excess of 12 per cent is allowed in the live load shearing and moment stresses for impact.

All web-stiffener angles are made with driving fit between flange angles, and all are crimped except at floorbeam and cross-

frame connections. Up to 50 ft. the end bearings on piers and abutments are made with flat plates fixed at one end and sliding at the other end. Cast-iron bed plates are used for spans of 45 ft. and less, under roller bearings, and for the fixed ends of roller-bearing spans where their heights are equal to the combined heights of the rollers and roller beds, so that the masonry heights are equal at both ends of the spans. Spans of 50 to 65 ft. have convex cast-steel hinge bearings to allow for deflection and preserve uniform pressure on the masonry seats. Up to 60-ft. spans, riveted steel plate bearings are also designed for alternate use where the abutments are of timber.

In the through-span floorbeam connections one of the vertical end web angles is shop-riveted to the main girder, and the

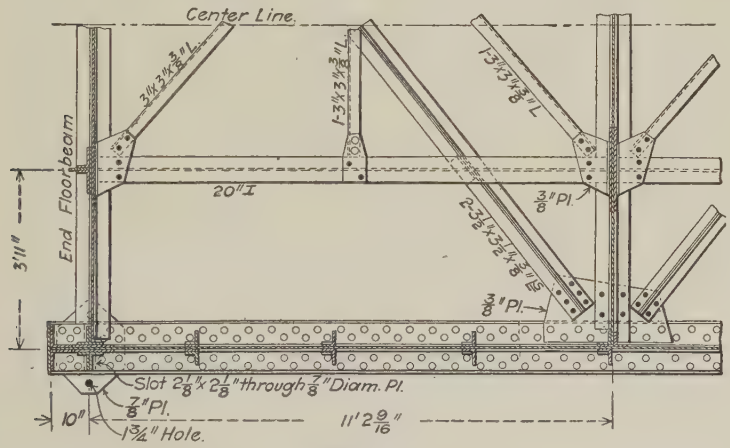


Fig. 53.

other is shipped loose, and both are field-riveted to the floorbeam rib, thus avoiding the necessity which otherwise arises of spreading the girders to assemble the floorbeams to them. This results in a justifiable increase of the minimum number of field rivets and in the omission of facing the floorbeam ends. Deck spans up to 60 ft. long are shop-riveted complete ; all other spans have the girders shipped separately or in sections. All spans of more than 65 ft. have 12-in. segmental rollers at one end.

All deck spans are made with square end girders with zigzag lateral angles field-riveted to horizontal plates field-riveted to the top and bottom flange angles. Transverse top and bottom struts and X-braces all composed of single angles are field-

riveted to the web-stiffener angles at the center points of alternate panels of the diagonal system. The web plates are reinforced over the end bearings, and at splices they have double cover plates with four vertical rows of rivets pitched closer at the ends than in the center. The end vertical web-stiffener angles have cover plates bent at right angles at the upper ends where they overlap and cover the ends of the flanges. The end sway brace frames are made much heavier than the intermediate ones.

In the 60-ft. deck spans the $\frac{3}{8}$-in. webs have an effective depth of 5.8 ft. The girders are $6\frac{1}{2}$ ft. apart on centers, and the flanges are made with pairs of full length $6 \times 6 \times \frac{3}{8}$-in. angles and a maximum thickness of $1\frac{3}{8}$ in. of $12\frac{1}{2}$-in. cover plates. The end bearings have concave shoes engaging convex pedestals which at the fixed ends are riveted to steel plates bolted to the masonry. At the sliding ends the pedestals are secured to the shoes by long vertical rocker bolts with countersunk ends, and are seated between flange guide strips on steel plates bolted to the masonry. The maximum shear is 126,158 lb., maximum moment 1,703,150 lb.

All through girders are made in three types, with both upper corners rounded, both square, or one square and one rounded according to their positions at the ends or in the intermediate spans of the bridge. The top flange angles are spliced in the tangent near the end so that the short bent angles are continuous for the end web-stiffeners. The girders are 13 ft. 2 in. apart on centers, and have pairs of 3×3-in. vertical web-stiffener angles on opposite sides of the web, dividing them into panels which are 3 ft. 4 in. long at the ends and 5 ft. long in the center. At panel points, 10 ft. apart, the web stiffeners consist of pairs of $5 \times 3\frac{1}{2}$-in. angles on the same side of the web between the outstanding 5-in. flanges of which are field-riveted the ends of the floorbeam webs.

The lower flanges of the floorbeams are nearly flush with the upper sides of those of the girders, with just sufficient clearance between them for a horizontal plate to be field-riveted to them and shop-riveted to the girder flanges and receive the field-riveted ends of the lateral diagonals which in all spans are single $3\frac{1}{2} \times 3\frac{1}{2}$-in. angles with the vertical flanges up. The vertical flanges are field-riveted to short oblique angle clips on the bottom flanges of the stringers. One angle is continuous in each panel, and the other one is field-riveted to the horizontal

plate which splices the ends of the other angle which is cut to clear it. The stringers are 15-in. 60-lb. I-beams with a pair of 6 × 6-in. vertical connection angles shop-riveted at each end to the stringer web and field-riveted with $14\frac{7}{8}$-in. rivets to the floorbeam web. The maximum floorbeam shear is 59,800 lb. in the floorbeams and 46,100 lb. in the stringers. The maximum total moments are 117,400 foot-pounds in the floorbeams and 80,900 lb. in the stringers.

Where there are two adjacent spans the adjacent ends of the girders have square upper corners which are connected by bent cover plates 12 in. long with their edges turned down so as to form flanges just covering the edges of the top flange angles and plates. They are field-riveted to one span and slide freely on the other span.

In the 60-ft. spans the maximum shear in the main girders is 129,500 lb., and the maximum total bending moment is 178,600 foot-pounds. The estimated weight of one complete span with masonry bearings is 69,900 lb.

The 100-ft. spans have cast-steel shoes bolted across the lower flanges at both ends which engage pins in full holes through their double webs and those of the corresponding pedestals, one of which is seated on a cast-steel bolster, and the other on a nest of four segmental rollers. The gusset plate kneebraces at the ends of the floorbeams differ from those of the 60-ft. span in that they do not reach quite to the top flange angles and in that their inclined edges are stiffened with pairs of 3 × 3-in. flange angles. They are separate from the floorbeam webs and are field-riveted to the floorbeam flanges and to the girder webs and stiffener angles. The girders are 8 ft. 6 in. high from back to back of flange angles and have maximum flange areas of two 6 × 8 × $\frac{3}{4}$-in. angles and $2\frac{1}{8}$-in. of 18-in. cover plates. The maximum shear is 210,250 lb., and the maximum total bending moment, allowing 8 per cent of rolling load for impact, is 1,381,250 foot-pounds. The estimated weight of a complete span is 173,600 lb.

Among the principal points elicited in the discussion of these standards were the following: that 8 × 8-in. flange angles would be better for the long spans; that the rivets in the horizontal surfaces of the top flange cover plates are an item of trouble and expense in fitting the ties for floor renewals; that the segmental rollers at the expansion ends of the longer spans were unnecessarily elaborate and expensive for the movements in-

volved; that the cost of reaming might be advantageously applied instead to increased weight of unreamed material and allow a difference in favor of the latter; that cast iron might be substituted efficiently for cast steel in the end bearings; that since it is impossible to exclude dirt from the roller bearings they should be made so that it will fall out as easily as it enters, a result which has only been attained with rail bed plates; that the segmental rollers may be made of cast iron with very little machine finishing, only on the bearing surfaces, and that they save expensive masonry and promote a satisfactory distribution of the load on the masonry; that the flanges are much better reinforced by horizontal than by vertical plates, and that flanges made with reversed pairs of angles are undesirable; that cast shoes, pedestals, and bolsters are much more satisfactory than riveted ones; that these standards can safely withstand an increase of 50 per cent in loading without undue overstraining of the metal, and that the plans have been carefully and conscientiously prepared, and that such a system of standards is good for a large railroad.

Chicago Great Western Railway Standards.

The accepted features of design of short span through plate-girder bridges on the Chicago Great Western Railway are illus-

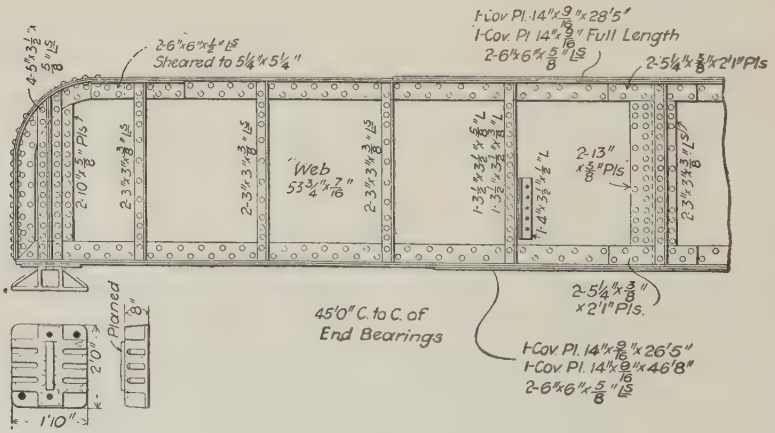


Fig. 54.

trated by the three 45-ft. spans of the single-track bridge No. 98 over the Wolf Creek.

It is made of medium steel with reamed rivet holes, and is proportioned for a train load of 4,000 lb. per lin. ft., preceded by two 154-ton consolidated locomotives. The girders are 54 in. deep and 46 ft. 8 in. long over all, and $14\frac{1}{3}$-ft. apart on centers. The webs are $\frac{7}{16}$ in. thick, made in three pieces each with the joints spliced with two vertical cover plates with four rows of rivets and two pairs of horizontal top and bottom flange web cover plates.

The maximum flange section is made with two $6 \times 6 \times \frac{5}{8}$-in. angles and two $14 \times \frac{9}{16}$-in. cover plates. The angles are full length, except that the top ones are cut about 3 ft. short at each end to splice with the end vertical web-stiffener angles of the

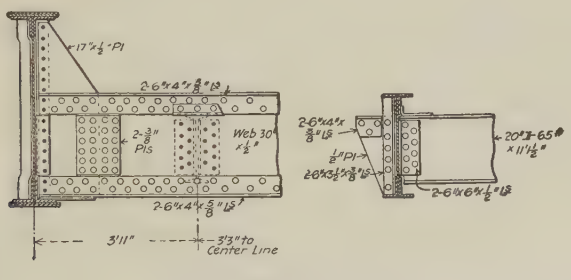


Fig. 55.

same size, which are curved around the corner. The joints are staggered about 8 in. on opposite sides of the web, and are spliced with cover angles and by the cover plate, which extends about 11 ft. back from the end of the girder, and is spliced with the projecting end of the second cover plate.

The web is divided into panels varying from 2 ft. long at the ends to 4 ft. at the center, by pairs of stiffener angles crimped except at the ends and at web splices. The end panels are reinforced by web cover plates and by vertical pairs of angles on the center line of the bearing. The end bearings are on sole plates projecting $4\frac{1}{2}$ in. beyond the edges of the lower flange cover plates, and each having two round or slotted holes for short $\frac{3}{4}$-in. bolts connecting them to the top flanges of the cast-steel pedestals which have transverse planed bearing surfaces 2 in. wide to receive the girder flanges, and allow them to deflect without producing eccentric or unequal stresses. The pedestals are seated on bed plates $\frac{3}{4}$ in. thick, and secured with $\frac{3}{4}$-in. anchor bolts 3 ft. long through both pedestal and bed plate.

The girders are connected by floorbeams 11 ft. $2\frac{9}{16}$ in. apart, which are made with their short end section of web extended beyond their top flanges and field-riveted to the vertical web-stiffener angles of the main girders and to short vertical web-connection angles opposite them. The heavy flange angles are all cut short on the same side of the floorbeam web to clear the vertical angles. The ends of the track stringer I-beams are coped to clear the vertical flanges of the top and bottom floorbeam flange angles, and the stringers are connected to the floorbeams by pairs of vertical angles shop-riveted to them and field-riveted to the floorbeam webs. In this case the track is on a curve, and the outer stringers are 24-in. and the inner ones 20-in. I-beams with their bottom flanges at the same level and their top flanges at different heights to correspond with the elevation of the outside rail, the ties being dapped evenly on them to an inclination of 4.78.

The 20-in. stringers have transverse bent plate connections riveted to their top flanges and to the floorbeam webs, which receive the zigzag horizontal angles riveted at their opposite ends to their webs at the center points. Each panel between girders and floorbeams is braced by a single zigzag diagonal made with a pair of angles riveted together back to back, and field-riveted to horizontal connection plates shop-riveted to the lower flanges of the girders and slotted to clear their web-stiffener angles. The diagonals are connected by angle clips to the bottom flanges of the stringers at their intersections.

The 80-ft. Kaw River bridge of the Chicago Great Western Railway at Kansas City, Kansas, is typical of the standard single-track through plate-girder construction for that railway. It is proportioned for the same load and governed by the same specifications as the short-span through bridges, and does not differ radically from them except in dimensions and materials. The floor system is the same as that described for the 45-ft. Wolf Creek bridge, except that as it is on a tangent both lines of stringers are 24-in. 80-lb. I-beams $13\frac{1}{3}$ ft. long, without transverse or zigzag bracing, and the floorbeams are 36 in. deep. The short end sections of the floorbeam webs are extended to form kneebraces cut short at the upper end to clear the vertical legs of the top chord flange angles, and stiffened with a pair of inclined flange angles on the outer edge, curved at the top, and having transverse oblique cover angles over the joints with the floorbeam top flange angles.

The girder webs are 84 in. deep, and have a uniform thickness of $\frac{1}{2}$ in. They are made in lengths of about 16 and 17 ft., spliced at each joint with two 14 × $\frac{3}{8}$-in. cover plates cut to clear the flange angles, and with two pairs of horizontal 7 × $\frac{7}{16}$-in. top and bottom flange cover plates. The flanges have a maximum section of two full-length 8 × 8 × $\frac{11}{16}$-in. angles and one 18 × $\frac{9}{16}$-in. and two 18 × $\frac{1}{2}$-in. cover plate, the first one being full length. Their expansion end bearings are the same as those for the 90-ft. deck span across the Wapsie River, but the fixed end bearings differ from those of that span in that they have a single tall cast-steel pedestal instead of a pedestal like that at the expansion end, and a separate bolster to compensate for the height of the rollers and bed plate.

In the standard short-span deck bridges the girders correspond to those of the short-span through bridges except that the upper corners are not rounded ; there are no end flange angles, the top flange angles are full length, and the track ties rest directly on the girder flanges. A 49-ft. 3-in. span is divided in seven panels by end and intermediate sway-brace frames, and the center frame is midway between two sets of web-stiffener angles. Each panel between sway-brace frames is braced with a single top and bottom lateral diagonal in the same plane, made with a pair of angles back to back with the vertical flanges of the upper angles turned down and those of the bottom angles turned up.

Chicago & Northwestern Railway.

Through-span plate-girder bridges of the Chicago & Northwestern Railway are illustrated by a deep-floor 80-ft. single-track span, proportioned for two 140-ton locomotives, followed by a train load of 4,000 lb. per lin. ft. The estimated weight of the span is 116,000 lb. exclusive of about 1,100 lb. of cast-iron pedestals. The material is structural steel, reamed, which is stressed to 9,000 lb. and 18,000 lb. net, for live and dead loads respectively. The maximum flange stress is 300,900 lb., and the gross area of the top flange is 40.48 sq. in., and the net area of the bottom flange is 34 sq. in. The end shear is 152,100 lb. Stringer flange stress and end shear 64,100 and 57,400 lb. Floorbeam flange stress 84,700 and shear 72,900 lb. The allowance for impact is 40 per cent minus 4 times the square root of the length loaded for maximum stress.

The girder webs are made in five 15½-ft. sections, spliced with four vertical rows of rivets at each joint, and having six additional flange splice rivets in horizontal 10-in. plates, top and bottom. Each intermediate section of the web is divided into two, and the end sections into three sections by pairs of crimped vertical web-stiffener angles. At the ends of the girders two pairs of web-stiffener angles and two narrow web reinforcement plates stiffen the girder over the bearing and give that portion of it an I-shape horizontal cross-section, virtually making a vertical post proportioned to transmit all the shear centrally to the cast-iron pedestal which distributes the load on the masonry with a pressure of about 266 lb. per sq. in.

The pedestals are rectangular castings with horizontal top and bottom plates connected by intersecting longitudinal and transverse webs about 7 in. apart on centers. The upper plate is planed, and is secured to the girder by two holes through its lower flange which are slotted 2 in. long at the expansion end. The base is planed and anchor-bolted to the masonry and is longer and wider than the top. Where the ends of girders of adjacent spans meet on the same pier they are seated on a single continuous pedestal 3 ft. long.

There are no vertical web-stiffener angles shop-riveted to the ends of the floorbeam webs. The webs are field-riveted to the web-stiffener angles of the main girders, and these rivets are driven through single end vertical angles on the opposite side of the floorbeam web which have a row of field rivets through their opposite flanges and the girder web. The angles are on opposite sides of the floorbeam web at the opposite ends, so that before they are assembled the floorbeam can be revolved horizontally to place without the necessity of spreading the girders. A similar arrangement is provided for the stringer and floorbeam connections. One of the two vertical connection angles at each end of each stringer is the regular web-stiffener angle of the floorbeam. Two opposite diagonal angles in the horizontal cross-section of each connection are the shop-riveted regular floorbeam web-stiffener angles. The other two angles are stringer web-stiffener angles and are field-riveted to both stringer and floorbeam.

The stringer flanges are notched to clear the floorbeam angles and beveled to clear the other angles so that the stringers can be horizontally revolved to position after the floorbeams are riveted in place. All of these angles have web

reinforcement plates on both stringers and floorbeam webs. The floorbeams are kneebraced to the girders with solid-web brackets reaching to the top flanges of the girders. The lateral system consists of single $3\frac{1}{2} \times 3\frac{1}{2}$ and 4×3-in. X-brace angles with the vertical flanges of intersecting diagonals turned in opposite directions to clear. Their ends are riveted to horizontal $\frac{3}{8}$-in. plates engaging the bottom flanges of both floorbeams and girders, and their vertical flanges are stayed to the stringer bottom flanges by short connection angles. The stringers have no other lateral angles and no sway-bracing. The end floorbeams have solid-web brackets on the outsides, in the planes of the stringers, which are virtually short extensions of the latter riveted to the vertical web-stiffener angles and long enough to carry a single-track tie, close to the floorbeam over the back wall of the abutment or the pier. The computed steel weight is 116,000 lb., pedestals 1,600 lb., one girder per lin. ft. 480 lb.

Recent deck-plate girder construction is illustrated in the design of a 55-ft. span calculated for a 140-ton locomotive which very nearly covers the full length of the span, and has four pairs of drivers spaced $4\frac{1}{2}$ ft. on centers, and each loaded with 40,000 lb. The assumed dead load per linear ft. is 450 lb. for track and 850 lb. for steel. The length of the span on centers of bearings is 54 ft. 2 in., and the effective depth is $5\frac{1}{2}$ ft. An allowance of 10.6 per cent is made for impact, and the maximum bending moment is 1,371,100 ft.-lb., which with a unit stress of 9,000 lb. requires a flange section of 27.7 sq. in. net. The gross areas of the flanges are both about 33.5 sq. in., and the net area of the bottom flange is 28 sq. in. Maximum end shear, including impact, is 115,800 lb. The bridge is made of "structural steel" reamed, and weighs about 46,900 lb. exclusive of about 1,400 lb. of cast-iron pedestals.

The flange angles are made in single pieces but differ in the top and bottom, the former being made with four angles, giving an H-shape cross-section especially designed for compression and unobstructed by rivets on the upper surface. This avoids notching the ties, and facilitates their removal and replacement. The bottom flange is the ordinary T-shape, suited for tensile stresses, and has two thicknesses of part length cover plates, while the top flange has no cover plates but is reinforced by web plates in the center. The web is made in three pieces spliced with ordinary double cover plates which do not extend

across the flange angles, and have four vertical rows of rivets. All of the web-stiffener angles terminate at the lower top flange angles to which they are riveted by short transverse angles. Smaller angles are fitted in the same lines between the top and bottom angles of the top flange. The end web-stiffener angles have fillers, the others are crimped over the vertical legs of the bottom flange angles. The web rivets in the flange angles are in staggered rows with a pitch of from 3 to $4\frac{1}{2}$ in.

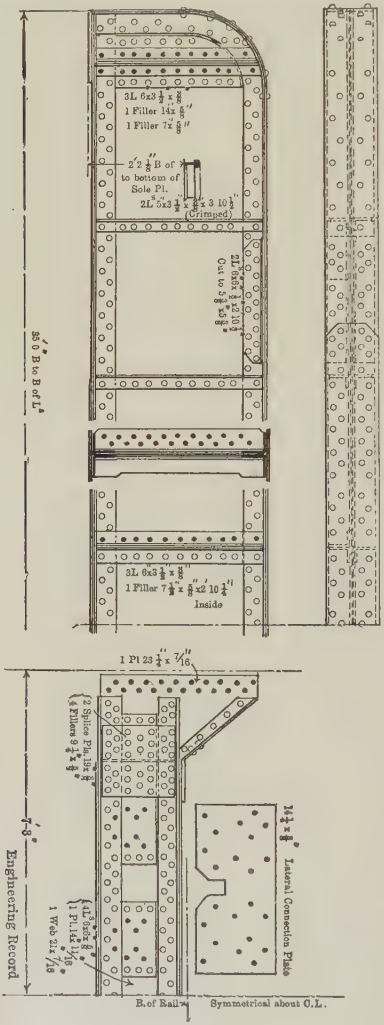


Fig. 56.

The bearings at both ends of the girders are on cast-iron webbed pedestals planed top and bottom and proportioned to reduce the masonry pressure to 300 lb. per sq. in. No difference is made between the fixed and expansion end pedestals, both of which are anchored. The girders have top and bottom lateral systems of $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$-in. angles with wide connection plates field-riveted to the flange angles and to the sway-brace frames. The upper system consists of X-braces with both flanges of the angles connected to the horizontal plates on the lower top flange angles. The lower system consists of zigzag angles with one flange connected to the horizontal plates. There is one center and two end sway brace frames, and there are six intermediate cross-strut frames, one at each panel point. All of them a reconnected to

both long and short vertical web-stiffener angles. The standard 45-ft. span corresponds to the 55-ft. span, except that the girders have an effective depth of only 4 ft. The gross flange section is 34.3 sq. in., and the steel weight is 34,700 lb.

The Illinois Central Railroad.

A large amount of important data for the computations, estimates, and detailing of bridges on the Illinois Central Railroad has been formulated and arranged by the Department of Bridges and Buildings, and issued to the office engineers and draughtsmen in a set of 7 × 11-in. blueprints bound and entitled "General Rules for Office Practice, Standard Diagrams, etc." They are designed to secure uniformity of design, and save a large amount of time in ordinary operations. They include systems of loading for truss and girder spans; unit stresses; sizes, pitch-gauging and staggering of rivets; instructions for the special features of pin-connected truss spans and for girder spans, and a large number of diagrams and tables.

Some of the data were prepared several years ago, and are now subject to revision and modification, but most of it conforms to present high-grade practice, or requires only slight modification to do so.

For deck spans the live load is scaled from diagrams of equivalent uniform loads corresponding to the train load. End shears are scaled from a diagram plotted from the compromise standard system of live loads for railway bridges. For through spans the floorbeam concentrations are computed for equivalent uniform live load plus dead load of floor system only, and these values plus the moment due to the dead weight of the girder give the maximum bending moments.

Unit flange stresses of 10,000 lb. and 14,000 lb. are allowed for single-track and double-track spans, respectively, and shears of 5,000 lb. and 7,000 lb. Shop rivets are allowed 16,000 lb. bearing and 8,000 lb. shear, and field rivets 10,670 lb. bearing and 5,335 lb. shear. Bed plates are allowed a bearing of 250 lb. per square inch on concrete, and from 300 to 350 lb. on stone. The pitch of rivets connecting flanges to web is determined by the formula $P = V \times D \div S$, where P is pitch, V is value of rivet, D is distance between rivet lines in top and bottom flanges, and S is shear.

Nearly all rivet data may be selected by inspection from

tables showing the length of shank necessary for required grips with round or countersunk heads ; the diagonal distances between rivets in staggered rows of different pitches ; minimum stagger for rivets in opposite flanges of angles ; single and double shear and bearing values for shop and field driven rivets ; different unit stresses for girders, floor systems, and laterals ; and lengths of bolts of different diameters required for various grips.

Double-track through spans are made with three lines of girders with I-beam floors for them, and single-track spans are made with I-beams 16 inches apart on centers web-connected so as to just clear the bottom flange angles. The girders are kneebraced at panel points by solid-web gussets riveted to the top flanges of the floorbeams, and the beams are covered with a tight floor of longitudinal $8\frac{3}{4}$-in. tongue and groove planks carrying 12 inches of ballast in which the ties are set. The floor planks are sloped upwards slightly at the ends of the floorbeams, where they rest on wedge-shape fillers, and every third plank is locked in position. Where solid-ballasted floors are not used the web plates of deck girders and of stringers in through spans project beyond the backs of the top flange angles, and engage slots cut in the ties locking them in position transversely. Lining ties with the long sides vertical are also notched over the girders or stringers every sixth tie on tangents and every fourth tie on curves, and the track is additionally secured by vertical bolts which engage guard timbers, ties, and top flanges of stringers. The steel weight of deck and through spans for class R loading is tabulated for standard length up to 100 ft. The weights of single and double track through and deck spans with and without ballast floors are also graphically given by a series of curves on cross-section paper of such dimensions that they can be scaled with approximate accuracy. The curves are plotted with ordinates and abscissæ obtained by formulas in which the weight x is given in terms of the square of the span y. For deck girders of 70 ft. or less, $x = 9.5\,y^2 + 200\,y + 450$. For spans of 70 ft. or over, $x = 28\,y^2 + 2{,}280\,y + 23{,}400$. For I-beam spans, $x = 35\,y^2 \div 352\,y + 1{,}215$. For through spans without ballast floor, when both girders are light, for spans of 76 ft. or under, $x = 1{,}824\,y - 26{,}120$; for spans of 76 ft. or over, $x = 75\,y^2 - 9{,}927\,y - 433{,}740$; for one light and one heavy girder for spans of 80 ft. and under, $x = 54\,y^2 - 1{,}460\,y - 18{,}100$; for spans of 80 ft. or more, $x = 90\,y^2 - 12{,}480\,y - 553{,}100$; for one heavy and two light girders for spans of 80 ft.

and under, $x = 4y^2 + 2{,}980y - 44{,}000$; for spans of 80 ft. or more, $x = 68y^2 - 7{,}100y + 352{,}800$. For through single-track bridges without ballast floors, the percentages of floor weight to total weight for different spans is as follows: for 40-ft. span, 57 per cent; 50-ft., 54; 60-ft., 51; 70-ft., 48; 80-ft., 44; 90-ft., 41; 100-ft., 38 per cent.

Design and computations are also facilitated by tables of decimals of a foot, clearances for through spans on tangents and curves, the latter being given as $816 + L \div 8R - 17D$, where L is length of span, R is radius of curve, and D is degree of curve.

Chicago, Milwaukee & St. Paul Railway.

On the 7,100 miles of railroad owned and operated by the Chicago, Milwaukee & St. Paul Railway Company there are approximately 114,000 lin. ft. of steel and iron bridges and viaducts, of which 76,000 lin. ft. are plate-girder spans. The latter are preferred for spans of 20 to 90 ft., and a large number are annually erected for new bridges and to renew old structures. During 1904 the total amount of steel bridge work erected on this system amounted to about 6,000 tons. Of this amount about 1,600 tons consisted of plate girders which were generally erected with derrick cars by the railway company bridge department.

Careful attention has been paid to the design of plate-girder spans, and the systematic observations and records of structures in service have resulted in the development of standard types that are used in most cases.

Standard 40-ft. through single-track spans are made with plate girders 52 in. deep, $38\frac{1}{2}$ ft. long c.c. bearings and 14 ft. 2 in. apart on centers. The ends of the girders are square over piers, at adjacent spans, and are rounded at the ends of the bridges, over abutments. They are made of full-length materials, without web or flange splices, except that the top flange angles are cut opposite each other on both sides of the web, about 5 ft. from each end and spliced to angles of the same size, curved 90 degrees, and continued down vertically to form end flanges reaching to the bottom flange and covered with a plate projecting under the end of the second top flange cover plate.

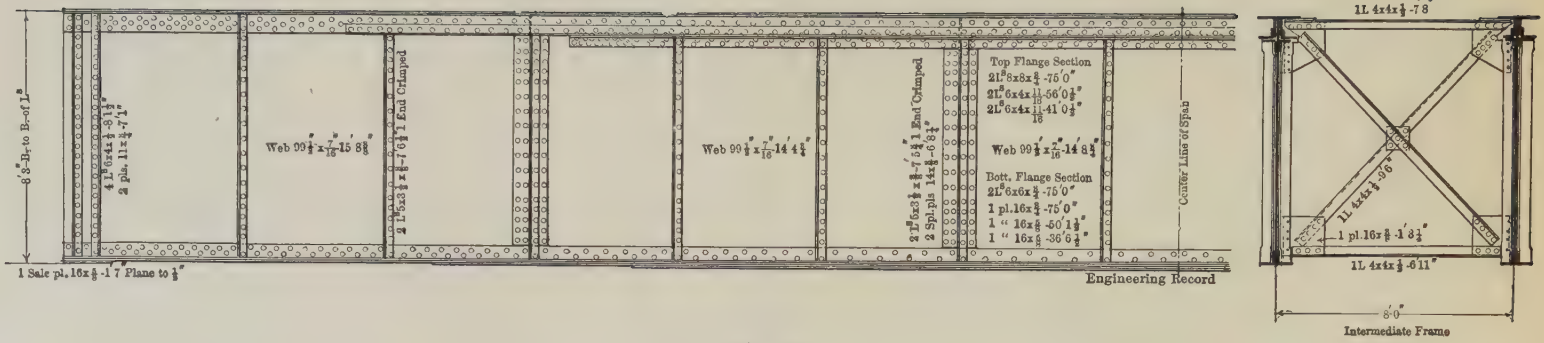
Double pairs of vertical angles stiffen the web at floorbeam connections, including those at the ends where they come over the centers of the bearings and receive the maximum shears;

at these points they are both field-riveted to the girder web; elsewhere one is field- and the other is shop-riveted to the web. They are separated $\frac{1}{2}$ in. to receive the floorbeam web and gusset connection plates, and are crimped over the vertical legs of the flange angles. The spaces between them are divided into panels from about 3 to $4\frac{1}{2}$ ft. long by single pairs of web-stiffener angles with fillers.

They have sole plates connected to cast-steel pedestals with bolts through round holes at one end and through slotted holes at the other end. There are five floorbeams, which, in class C3, for a shallow floor, are only $21\frac{1}{2}$ in. deep with $\frac{1}{2}$-in. webs, and a pair of $6 \times 6 \times \frac{5}{8}$-in. angles and a $14 \times \frac{1}{2}$-in. cover plate for each flange. The end sections of the web project above the top flanges, and are shop-spliced with cover plates to the middle section, and field-riveted between the girder web-stiffener angles. The stringers are four lines of 15-in. I-beams. Each panel has $6 \times \frac{3}{8}$-in. X-brace lateral bars riveted to the lower flanges of the stringers and to horizontal connection plates notched to clear the web stiffeners, and field-riveted to the floorbeam and girder lower flanges. The C4 type differs from the C3 type in that it has two lines of plate-girder stringers 8 ft. apart on centers.

All material except pedestals is soft steel, and all shop rivet holes are punched $\frac{3}{4}$ in. and reamed to $\frac{15}{16}$ in. Field rivet holes are punched full size except for floorbeam and stringer connections, which are punched $\frac{3}{4}$ in. and reamed to a drilled iron template 1 in. thick.

Deck plate-girder single-track spans, 70 ft. or more in length, conform essentially to the features illustrated in the accompanying typical engravings from a drawing dated January, 1904. The top flanges are made without cover plates and have full-length $8 \times 8 \times \frac{3}{4}$-in. angles, reinforced by a pair of reversed 6×4-in. angles riveted to the lower edges of their vertical legs in the middle of the span and by somewhat shorter 6×4-in. angles riveted to them, back to back. This arrangement leaves the upper surface of the flange unobstructed by rivet heads and provides a good seat for the ties which are all framed alike and held in position transversely by being notched over the web which projects $\frac{3}{4}$ in. beyond the backs of the flange angles. The bottom flanges are made with a single pair of 6×6-in. angles reinforced by one full-length and several shorter 16-in. cover plates. The web-stiffener angles are crimped over the flange angles except at splices and at the end bearings where


6'-3" Dr. to B. of L.s
4 L.s 6x4x½ -8'1½"
2 pls. 11x½ -7'1"
Web 99½" x 7/16 - 15' 8 8/16"
2 L.s 5x3½x½ -7'0½ 1 End Crimped
Web 99½" x 7/16 - 14' 4¼"
2 L.s 6x3½x½ -7'0½ 1 End Crimped
2 Spl. pls 14x½ -6'8¼"
Top Flange Section
21 L.s 8x8x¾ -75'0"
21 L.s 6x4x 11/16 -56'0½"
21 L.s 6x4x 11/16 -41'0½"
Web 99½" x 7/16 - 14' 8¼"
Bott. Flange Section
21 L.s 6x6x½ -75'0"
1 pl. 16x¾ -75'0"
1 " 16x⅝ -50'1½"
1 " 16x⅝ -36'6½"
Center Line of Span
1 Sole pl. 16x⅝ -1'7" Plane to ½"
Engineering Record

1L 4x4x½ -7'8"
1L 4x4x½ -9'6"
1 pl. 16x⅝ -1'3½"
1L 4x4x½ -6'11"
8'-0"
Intermediate Frame

Fig. 57.

there are two pairs in line with the transverse webs of the cast-steel shoes.

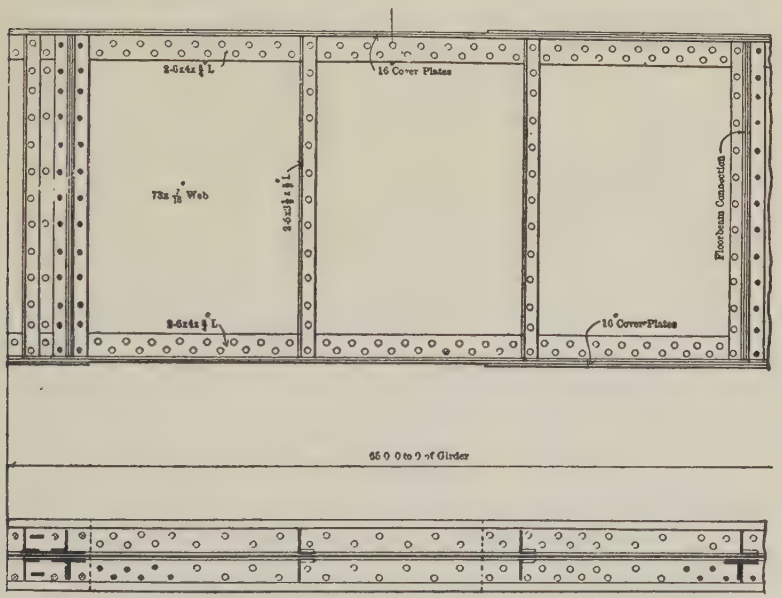
The shoes and pedestals are locked together by pins engaging full holes in the outside webs and half holes in the center webs. The shoes are bolted to the lower flanges, and the pedestals are seated at the fixed ends on cast bolsters and at the expansion ends on segmental rollers with rail beds. The girders are connected at alternate panel points by transverse sway-brace frames field-riveted to the vertical web-stiffener angles, and by zigzag lateral angles in the plane of the top and bottom flanges. All laterals are field-riveted to connection plates field-riveted to the girder flanges; the top flange plates being also field-riveted to the sway-brace frames; the bottom connection plates clear the sway frames. All connection plates have two rows of flange rivets and are notched to clear the web-stiffener angles where necessary. At the ends of the span they have short angle clips to connect them with the vertical flanges of the lateral angles.

In a 75-ft. span the 99½-in. web is $\frac{7}{16}$-in. thick throughout, and is made with four joints, each spliced with four vertical rows of rivets through two 14 × ⅜-in. splice plates 7 ft. long which also act as fillers for a pair of web-stiffener angles. The maximum top flange section consists of two 8 × 8 × ¾-in. angles 75 ft. long, two 6 × 4 × $\frac{11}{16}$-in. angles 56 ft. long, and two 6 × 4 × $\frac{11}{16}$-in. angles 41 ft. long. The bottom flange has two 6 × 6 × ¾-in. angles 75 ft. long, one 16 × ¾ in., one 16 × ⅝ in., and one 16 × ⅝-in. cover plate respectively, 75, 50, and 36½ ft. long. The four intermediate sway-brace frames are made with single 4 × 4-in. angles except their top struts which are made with two 4 × 4-in. angles, back to back. The end frames are made with 6 × 4-in. diagonals, and the lateral diagonals are single angles varying from 4 × 4 × ½ in. to 6 × 6 × ⅝ in.

The 75-ft. through span corresponds very closely with the 40-ft. through span, and the girders are made with a 91½-in. web plate ⅜ in. thick through, with two shop-riveted splices. The flange angles are 6 × 6 × ¾ in., and their cover plates are 16 × ¾ and ⅝ in. The panels are about 5 ft. long, and the flange angles are made in single lengths except as spliced for the curved end and corner pieces. The bearings are 73 ft. 6 in. apart on centers, and have riveted to the bottom flange a planed sole plate ¾ in. thick and 19 in. long, and of the same width as the flange. This plate is connected by four 1¼-in. bolts, with a triple web cast-steel shoe having a 4-in. pin engaging a cor-

responding pedestal seated at the fixed end on a webbed bolster, and at the expansion end on four segmental 8-in. rollers, and a bed plate made of 75-lb. rails, planed down to a height of $4\frac{1}{2}$ in. and riveted to a $24 \times 1\frac{3}{8} \times 36$-in. base plate with four $1\frac{1}{2}$-in. anchor bolts.

The floorbeams are 39 in. deep, and the riveted stringers $14\frac{1}{2}$ ft. long are made with a $2\frac{1}{2} \times \frac{1}{2}$-in. web, and four $6 \times 4 \times \frac{3}{4}$-in. flange angles with eighteen web-connection rivets and a shelf angle at each end. The top flanges are several inches below those of the floorbeams, and the web projects $\frac{3}{4}$ in. above them to engage a slot in the track ties. The lateral system consists of single $6 \times 4 \times \frac{3}{8}$-in. angle X-braces, field-riveted to horizontal plates field-riveted to the lower flanges of the girders

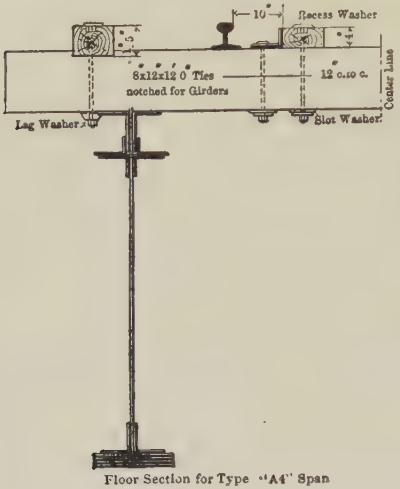


Fixed End of Sixty-Five Foot Span, Type C4.
Fig. 58.

and to the stringers by angle clips on their vertical flanges. All angles have their vertical flanges up; one in each panel is made continuous, and the other is cut to clear and spliced at intersection with a horizontal flange plate, and short angles splicing the vertical flange to correspond with the end connections.

Shallow-floor through plate-girder bridges of spans up to 100 ft. have girders of a standard design corresponding in detail and

general dimension to the 35-ft. span Standard C3 type, of which an illustration is here given. It is made entirely of soft steel with rivet holes punched $\frac{3}{4}$ in. diameter and reamed to $\frac{15}{16}$, those for the floorbeam connections being reamed with iron templates 1 in. thick.

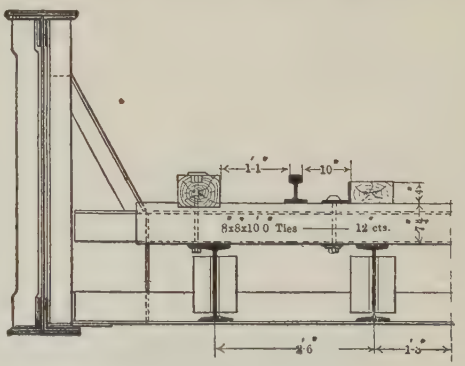


Floor Section for Type "A4" Span

Fig. 60.

The girder has a single full-length 46 × $\frac{3}{8}$-in. web plate, and is cambered $\frac{1}{4}$ in. by a slight displacement of the templates when the steel is laid out, thus easily giving the required camber to the vertical legs of the flange angles.

Care is taken to space the top and bottom flange angles $\frac{1}{2}$ in. farther apart, back to back, than the width of the web plate, so that there is a $\frac{1}{4}$-in. clearance between the edge of the web plate and the flange cover plates. In intermediate span girders the top and bottom flanges have single-piece angles and cover plates. The upper corners of the girder are square except at the abutment ends, where they are rounded as shown in the part elevation. At the square ends the end web-stiffener angles are of the same size as the intermediate web stiffeners, but are set back about 3 in. clear of the end of the web. The intermediate web-stiffener angles are crimped except at floorbeam connections, where they have fillers, and one angle of each pair is field-riveted to both girder and floorbeam, the other angle being shop-riveted to the girder and field-riveted to the floor-

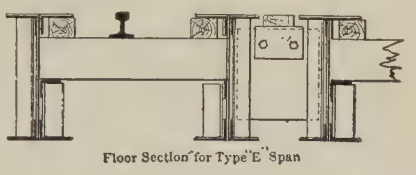


Floor Section for Type "C3" Span

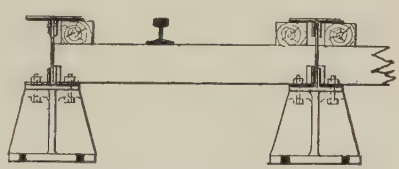
Fig. 61.

beam. The span is divided into three 10-ft. 11-in. panels by the floorbeams, which for this type of structure are very shallow and heavy.

The panels between floorbeams are X-braced by 6 × ⅜-in. diagonal bars with four rivets at each end to ⅜-in. connection plates notched to clear the web-stiffener angles and field-riveted to the bottom flanges of the floorbeams. They also have two field rivets at all intersections with each other and with the lower flanges of the stringers. There are two lines of 15-in. 60-lb. I-beam stringers for each rail. Track ties 8 × 8 in. by 10 ft. long, 12 in. apart on centers, are carried on the top flanges of the stringers and are bolted to the outer lines of stringers only. There are two guard rails and a guard angle for each rail, arranged as shown for type C4, but spaced somewhat differently. The end ties are carried over the bridge seats by stringer brackets made with short sections of 15-in. I-beams.



Floor Section for Type "E" Span



Floor Section for Shallow Girders

Fig. 62.

The standard type C4 single-track through 65-ft. span girder differs from the span already described chiefly in the floor construction, which has deeper floorbeams and only two lines of stringers, which are riveted plate girders 12 ft. 6½ in. long with a 22½ × ½-in. web and pairs of 6 × 4 × 11/16 in. flange angles without cover plates. Each panel is X-braced with 4 × 4 × ⅜-in. angles, one of which is continuous, and the other is cut to clear it at the intersection and spliced with a horizontal bottom flange plate. Short reversed angles connect the vertical flanges of the diagonals to the bottom flanges of the stringers, and care is taken to develop the strength of both flanges of the diagonals by short angle clips riveted to the vertical flanges at all connections.

The floorbeams have a 38¾ × ½-in. web and pairs of 6 × 4 × 11/16-in. flange angles with cover plates. The thickness of the web is made uniform throughout so as to avoid splicing the ½-in. end sections to thinner center sections. Their end gusset

plates and girder connections correspond to those of the 35-ft. span, except that the web splice plates are extended to increase the bearings of rivets in the stringer connections. The web plate is made in lengths of 21 and 22 ft. spliced with $\frac{1}{4}$-in. clearance at the joints which have four vertical rows of rivets through a pair of $13\frac{1}{4} \times \frac{3}{8}$-in. cover plates. The sole plates take bearing on webbed cast-iron pedestals 10 in. high with 16 × 19-in. tops and 25 × 30-in. bases, seated on sheet lead. The holes in the girder flanges are slotted at both ends of the girder so that both ends can slide on the pedestals. Where the corners of the girders are rounded and the top chord angles are continued vertically to the bottom chord angles, the ends of the bottom flange angles are fitted against the end cover plate, and the lower end of the top flange angles are fitted against the bottom flange angles.

Standard type A4 for long-span deck girders is illustrated by the 75-ft. single-track span here shown in part elevation. The most notable feature is the special detail of heavy top flange section which has been adopted by the railway company several years ago. It is without web or cover reinforcement plates and is composed entirely of angles, giving maximum compressive stiffness and arranged to leave the upper surface of the flange flush throughout to receive the track ties without framing or cutting to clear rivet heads. The double line on the upper side of the top angle looks, in elevation, like a cover plate, but is really the elevation of the upper edge of the web plate which in this type of girder is specified to project $\frac{3}{4}$ in. above the backs of the flange angles. The ties are easily notched with transverse saw cuts to engage it, and are thus secured against lateral displacement. The same device is applied to all plate-girder floor stringers.

The span is divided into panels of about $6\frac{1}{2}$ ft. by transverse sway frames like the one shown in the cross-section, and by the two end frames which are similar but have 6 × 4 × $\frac{1}{2}$-in. diagonal angles, and by transverse angle struts between the sway frames. The upper heads of the rivets connecting these frames to the top flange angles of the girder are countersunk so as to present no obstruction for the ties. Every panel of the top and bottom flange is braced by a single lateral angle of 4 × 4 in. and 6 × 6 in. maximum size in bottom and top systems respectively.

The 8 × 12-in. ties 12 ft. long and 12 in. apart on centers are

secured at each end with a bolt and a lug-washer to the top flange of the girder, as shown in the floor detail. The pieces of 4 × 8-in. inner guard timbers are bolted at each end and in the center, and are spiked to all the intermediate ties and spliced at joints. The 6 × 8-in. outer guard timbers are notched over the ties, bolted to every third tie and spiked to the intermediate ties. A clearance of $\frac{1}{4}$ in. is left between the 6 × 4 × $\frac{1}{2}$-in. guard angles, and they are bolted to every third tie. These fastenings are the same for the floors in types C3 and C4.

The girders are proportioned for a dead load consisting of the steel weight of the span plus a track weight of 450 lb. per lin. ft., and for a live load consisting of two consolidation locomotives coupled, each with 50,000 lb. on each of four axles, 5 ft. apart, as given in Cooper's class E-50, or for a special load of 130,000 lb. on two axles, 7 ft. apart, both loads being followed by a train load of 5,000 lb. per foot except in ore-carrying lines, where the train load is increased to 7,000 lb. The maximum unit stresses allowed are 8,000 lb. for riveted and I-beam stringer flanges, 8,500 lb. for net section of bottom flanges of the floorbeams. The allowed unit stress on the net section of the bottom flange of main girders varies uniformly from 8,000 lb. for 20-ft. spans, to 9,000 lb. for 100-ft. spans.

The shop drawings of all bridges are made in the office of the bridge engineer, so that in some respects the specifications do not need to be so specific as if intended to govern outside designers, but the work is much facilitated and uniformity is insured by very complete draftsmen's rules, tables and standard details, which save a large amount of repetition in routine work. All tracings are made to one of two standard sizes, 1 ft. 11$\frac{1}{2}$ × 2 ft. 10$\frac{1}{2}$ in. or 8$\frac{1}{2}$ × 14 in., and the riveting, clearances, dimensioning, arrangement of figures, notes, titles, and other general features of the drawings are carefully specified. There are tables for the values of shop and field rivets for general work, flanges and floor system, lengths of field-rivets and sizes of their heads, pitch, gauge and stagger of rivets, multiples of pitch for rivet spacing, end shears, maximum bending moments and maximum floorbeam reactions for one rail under Cooper's E-50 loading. Standard details are also provided for the ties, guard rails, and timbers in different types of floors, for pedestals, shoes, bolsters, rollers and bed plates, and diagrams are given of typical abutments with dimensions varied for different conditions. All designs and construction are under the direc-

tion of Mr. C. F. Loweth, engineer and superintendent of bridges and buildings, by whom most of the present plate-girder standards have been prepared or modified.

New York Central & Hudson River Railroad.

The standard designs of the New York Central & Hudson River Railroad for 1904 provide for long and short span deck and through plate-girder bridges, which are made with sections and dimensions to suit the given conditions, but are uniform in general character, arrangement and details. They are all made with soft steel, with holes punched and reamed, and, where the metal is $\frac{3}{4}$ in. or more in thickness, drilled from the solid. No metal is less than $\frac{3}{8}$ in. thick. The depths of the girders are usually from $\frac{1}{8}$ to $\frac{1}{12}$ their span. The flange angles are made with equal legs, and have two staggered rows of rivets through the web, and are reinforced by one or more cover plates of equal section in top and bottom flanges. The girders are divided into panels not longer than the depth by pairs of vertical web-stiffener angles fillered flush with the flange angles and milled at both ends ; and the end panels have additional stiffeners to provide for the shear above the end bearings.

In deck spans the girders for each track are placed generally $6\frac{1}{2}$ ft. apart on centers, and are connected by vertical transverse sway-brace frames, and by top and bottom zigzag lateral diagonals in the panels between the sway braces. The end sway braces are made with pairs of horizontal angles for the top and bottom struts, and with single X-brace angles riveted together at intersections and to the web-stiffener angles with connection plates shop-riveted to the angles and field-riveted to the girders. Intermediate frames differ from them only in having single top and bottom horizontal angles. The lateral angles are riveted to horizontal connection plates engaging the girder flanges and the sway-brace frames. The upper corners of the girders are square, and are exactly flush with the back walls of the abutments, on which and on the top flanges the cross-ties are seated.

The adjacent girders for two-track spans — that is, the two center girders in sets of four — are connected together by top and bottom horizontal transverse struts in the planes of the sway-brace frames, but in the panels thus formed diagonals are omitted to avoid transmitting shear from a loaded to an unloaded girder. In long spans shoes are riveted to the ends of

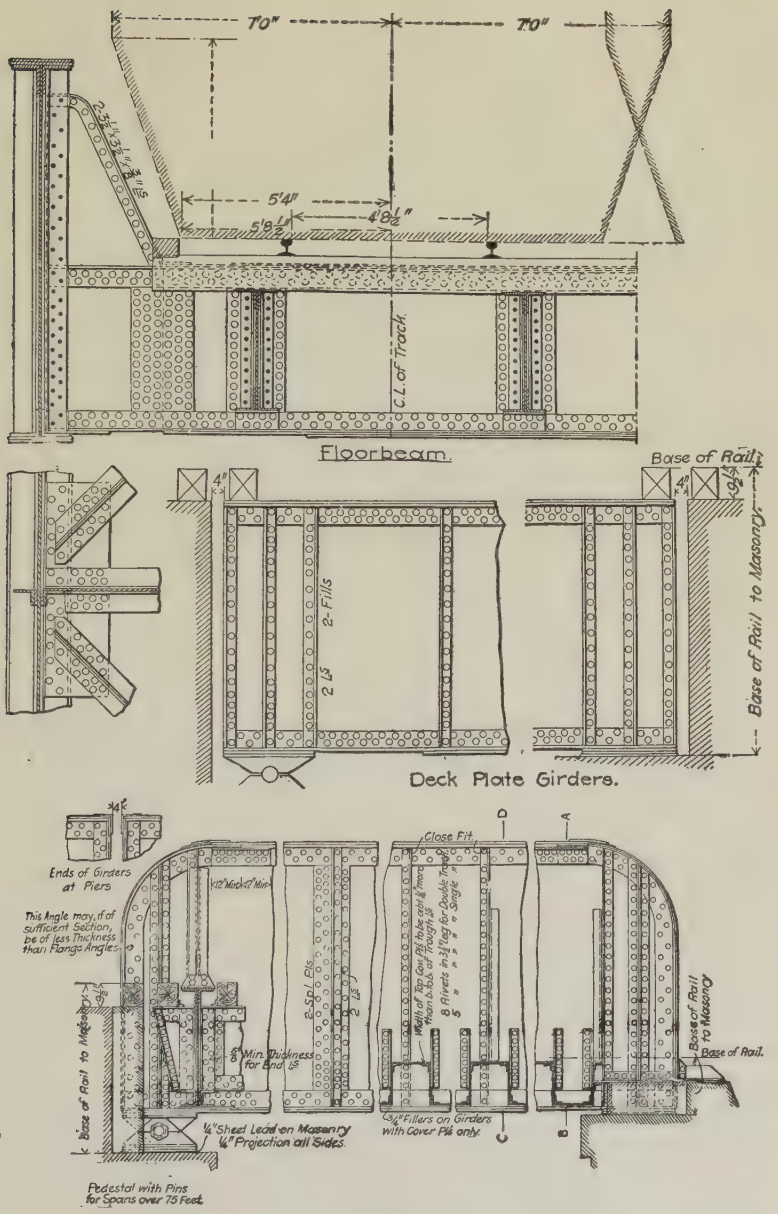

70"
70"
5'4"
4'8½"
5'8½"
C.L. of Track.
Floorbeam.
Base of Rail
Base of Rail to Masonry
2-Fills
Deck Plate Girders.
Ends of Girders at Piers
This Angle may, if of sufficient Section, be of less Thickness than Flange Angles
Close Fit
Base of Rail to Masonry
Base of Rail.
Pedestal with Pins for Spans over 75 Feet.

Fig. 63.

the lower flanges and have pin bearings in pedestals similar to those for short truss spans. At the fixed ends the pedestals are anchor-bolted to the masonry, and at the expansion ends they are seated on nests of rollers with center guide ribs. The rollers are set on cast-steel cellular bolsters with longitudinal grooves in their upper surfaces, and the shoe base plates extend beyond the roller ends with slotted holes to engage anchor bolts through the bolsters. Both ends of short-span girders have bearings on sole plates riveted to the bottom flanges and seated on bed plates. Both sole plate and bed plate are secured to the

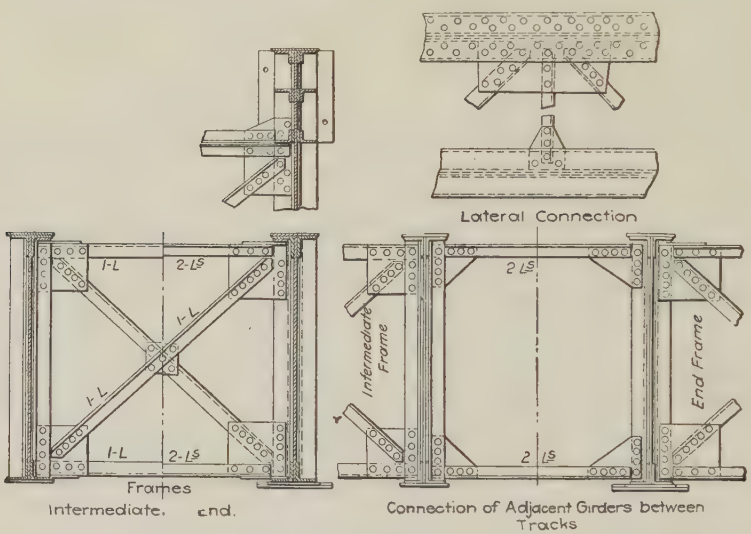


Fig. 64.

masonry by two anchor bolts at opposite diagonal corners, but at the expansion end the bolt holes are slotted and the sole and bed plates are planed.

Through girders have the upper corners rounded at the ends of the bridge, and square elsewhere. Where the corners are rounded the top flange angles are cut off square about 12 in. from the point of tangency of the curve, and the short bent angles of the same size but less thickness are riveted on tangent to them at their upper ends, and forming flanges for the ends of the webs. Both legs of these angles are spliced to the main flange angles with inside round-root angles, and they have a thin cover plate of equal width with the main flange cover plate, the butt joint between the two being fillered and covered as if

it had to endure positive stress. Adjacent ends of through girders in the same line have square upper corners with vertical end flange angles smaller than the flange angles, and have a vertical end cover plate as wide as the flange cover plate, but thinner. The upper corner joint is covered by a rectangular bent plate.

Through spans are of two types, with plate-girder floorbeams and stringers, and bottom lateral X-bracing, or with solid steel trough floor and no lateral bracing. The floorbeam webs are

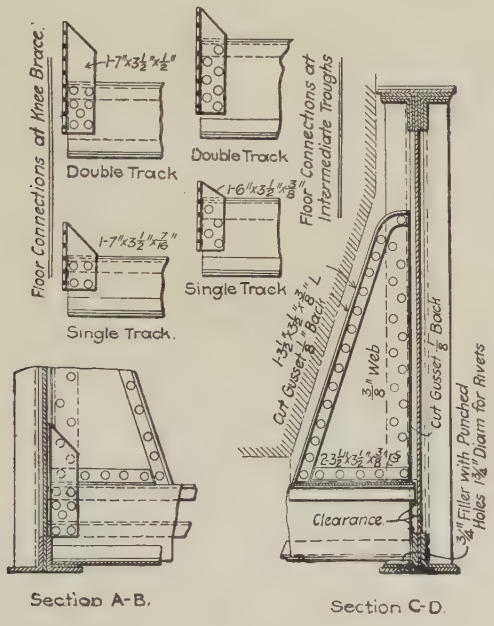


Fig. 65.

made in three sections shop-spliced with double cover plates and six vertical rows of rivets. The end sections project beyond the tops of the floorbeams nearly to the top flanges of the girders to afford field-riveted connections with the vertical web-stiffener angles, and to form solid kneebraces with the inclined edges stiffened with a pair of $3\frac{1}{2} \times 3\frac{1}{2}$-in. flange angles, and having a bent cover-plate connection with the top flange of the floorbeam.

The floorbeam webs are reinforced for the field-driven rivets of the stringer connections. The stringers are made of such depths that their flange angles just clear the vertical legs of the

floorbeam flange angles. They are seated, for convenience in erection, on short horizontal angle clips shop-riveted to the lower flange angles of the floorbeams. The 8 × 10-in. ties, 12 ft. long and 12 in. apart on centers, are laid on edge and dapped ½ in. over the stringers. Their depth is sufficient to carry the rails clear to the top flanges of the floorbeams. Short cantilever brackets are riveted to each end floorbeam in the planes of the stringers to reach nearly to the back wall of the masonry and support one track tie between it and the floorbeam.

The solid-floor type has rectangular troughs with their bottom plates fillered up from the lower flange angles of the girder. The ends of the troughs are connected to the girder webs with vertical angles having one long row of rivets in the web and two short staggered rows in the side plate of the trough. The girder bearings are seated in longitudinal recesses in the substructure masonry, and the spaces around them are covered by horizontal apron plates riveted to the end of the girder web and to the end trough, and overlapping the back wall with a bearing on its upper surface.

Solid floor spans have no lateral bracing, but stringer spans have lower zigzag lateral diagonals in the panels between floorbeams. They consist of single or double angles riveted to horizontal connection plates engaging the lower sides of the bottom flanges of the floorbeams and the upper sides of the girder flanges. The respective weights of 50-ft. deck, floorbeam-through and solid-floor-through single-track spans are about 44,000, 63,000, and 85,000 pounds. For 75-ft. spans of the same kinds they are about 84,000, 115,000, and 150,000 pounds.

Canadian Pacific Railroad.

The standard single-track plate-girder spans for the Canadian Pacific Railroad are of several different types according to the length and conditions, position, headroom, etc. Deck bridges of 30-ft. clear span have two girders 36 in. deep, 15 in. wide, 33 ft. long, and 9 ft. apart on centers with flat plate bearings 31 ft. apart on centers. All plates and angles are made in full-length pieces so that there are no splices. The 6 × 6 × ½-in. flange angles have two staggered rows of rivets in each flange. Pairs of 3 × 3-in. crimped vertical intermediate web-stiffener angles with a driving fit between the flange angles divide the girder into panels about 5¼, 4 3¼, and 2¾ ft. long, counting from the

center. The 14-in. end panel is formed by two pairs of 6 × 3½ in. angles which virtually form an end post I-shaped in horizontal cross-section, and calculated to resist the maximum shear. Each flange has 15-in. cover plates, one of them ⅜ in. thick and 33 ft. long, and the other $\frac{7}{16}$-in. thick and 22 ft. 4 in. long.

The girders are connected together by two end and two intermediate sway-brace frames 3 ft. deep, and riveted to the web-stiffener angles. They are light lattice girders with all members made with single angles having gusset-plate connections. Single 3 × 3-in. zigzag lateral diagonal angles are riveted across the panels in the opposite directions in the planes of the top and bottom flanges. Sole plates are riveted to the bottom flanges and take bearing on planed steel bed plates at the fixed and sliding ends. Each span is completely assembled, open holes reamed, and the separate parts match-marked in the shops. Track ties 12 × 14 in. × 11 ft. long and 16 in. apart are laid across the top flanges, and are countersunk to clear the rivet heads.

A half-deck bridge of 60-ft. clear span has girders 6 ft. deep, 65 ft. 4 in. long, and 13 ft. apart on centers, with spherical bearings 63 ft. 1 in. apart on centers. The web is made in four pieces shop-spliced with two 12 × $\frac{5}{16}$-in. cover plates, having four vertical rows of rivets at each joint. Each flange angle is made with two pieces with joints about 8 ft. from the center of the girder, staggered on opposite sides of the web, but opposite each other in top and bottom flanges and spliced with cover angles. No provision is made for splicing the web plate in the flanges. The web stiffeners are from 3 to 5 ft. apart, and consist of pairs of angles at alternate points and single outside angles at the intermediate points. All of them are crimped and have driving fits against the flange angles. The first flange cover plates are made in two pieces with a joint at the center of the girder, the second ones are also made in two pieces with joints about 3 ft. from the center of the girder; both these joints are opposite and duplicate in top and bottom flanges, and in each flange both joints are spliced by the outside cover plate 12 ft. long. At the ends of the bridge the top flanges are rounded, and the angles are continuous to their intersection with the end vertical angles. Where two girders of adjacent spans meet, their ends are square.

The girders are connected by two end and five intermediate sway-brace frames made with pairs of horizontal angles 27 in.

apart on centers, and single X-brace angles. These frames have wide vertical connection plates at the ends, which project beyond their top flanges to form kneebraces reaching to the top flanges of the girders and field-riveted to the web-stiffener angles. They have about 2 in. clearance from the lower flanges of the girders and from the rail base, and their end plates are notched to clear the shelf angles on the girder webs 21 in. above the bottom flanges, to which are riveted horizontal connection plates for the 3 × 3-in. X-brace lateral angles. These have their vertical flanges turned down; one of them is continuous in each panel, and the other is cut to clear it and spliced across the intersection with a horizontal flange plate shop-riveted to both flanges and field-riveted to the continuous angle. There are no floorbeams or stringers, but there is a continuous 5 × $3\frac{1}{2}$ × $\frac{5}{8}$-in. horizontal angle riveted to each girder web to support the ends of the 12 × 15-in. track ties 16 in. apart on centers. The span is completely assembled, reamed, and match-marked in the shop.

The girders for the 100-ft. standard through span are 105 ft. $10\frac{1}{2}$ in. long, $8\frac{1}{2}$ ft. deep, and $18\frac{1}{2}$ ft. apart on centers, with spherical bearings 103 ft. $1\frac{1}{2}$ in. apart, fixed on riveted pedestals at one end, and seated on segmental rollers at the other end. The web plates are in sections about 10 ft. long, and are spliced with double cover plates extending only to the flange angles and having four vertical rows of rivets. The upper corners of the webs are rounded, and the top flanges are correspondingly curved downwards for a distance of 16 ft. from each end so as to reduce the lengths of the end web-stiffener angles to 5 ft. $3\frac{1}{2}$ in. Vertical web stiffeners are placed 8 ft. $7\frac{1}{8}$ in. apart, and consist of pairs and alternate pairs of 5 × $3\frac{1}{2}$ × $\frac{3}{8}$-in. angles. An extra stiffener is placed in the middle of the end panel, another is placed over the center of each bearing, and a third about 16 in. from it.

The 8 × 8 × $\frac{5}{8}$-in. flange angles are made in two lengths with staggered joints directly opposite each other in top and bottom flanges, spliced with 8 × 8 × $\frac{11}{16}$-in. angles, cover angles planed to $7\frac{3}{8}$ × $7\frac{3}{8}$ in. The flanges have five 19-in. cover plates aggregating $3\frac{1}{8}$ in. in thickness. The first one reaches from end to end of the girder, and the others are shorter with their joints spliced by the overlapping ends of the outer plates. Each leg of the flange angles has two staggered rows of rivets usually pitched $2\frac{7}{8}$ in.

The girders are framed with $\frac{3}{4}$ in. camber and are connected by plate-girder floorbeams, field-riveted to the double angle stiffeners 17 ft. $2\frac{1}{4}$ in. apart. Their lower flanges are field-riveted to horizontal connection plates shop-riveted to the upper sides of the girder lower flange angles, and receiving the $5 \times 3\frac{1}{2}$ in. lateral X-brace angles. The floorbeam webs are made in three sections, the center sections being cut 21 in. short of the centers of the girders. The flange angles project beyond them, forming jaws which engage and are field-riveted to gusset plates shop-riveted between the web-stiffener angles, and forming at the same time the end sections of the floorbeam webs and knee-braces integral with them, and spliced to the center sections of the web with double cover plates.

At the intermediate floorbeams the top flanges of the girders are braced by pairs of inclined angles riveted to connection plates on the outside web-stiffener angles. The end sections of the floorbeam webs project through slots in the girder webs, and their corners are riveted between the vertical flanges of the outside brace angles near their center points, thus making wide and rigid sway-bracing integral with the floorbeam webs. The slots for the transverse plates come at the joints of the girder webs so that the latter merely require to be slightly notched on the edges. The track is carried on two lines of 20-in. I-beam stringers, and there are two additional lines of 15-in. outside stringers which have their lower flanges riveted to the lateral angles.

Union Pacific Railroad.

The Union Pacific Railroad Company has adopted standard designs of plate-girder construction for single-track bridges of 10 to 100-ft. spans. The deck bridges are marked Common Standard, adopted May, 1904, and are fully detailed for spans of 20-30-40-50-60-70-80-90 and 100 ft.

The 20-ft. spans are made with three lines of 20-in. 90-lb. I-beams under each rail. The beams are shop-riveted solidly together by stiffened vertical transverse diaphragms at the ends, and at two intermediate equidistant intermediate points, and by zigzag top lateral angles between the inner beams. The end bearings have $8 \times \frac{3}{4}$-in. sole plates riveted across the lower flanges of all beams and anchored to the abutment masonry with two $1\frac{1}{4}$-in. rag bolts 12 in. long.

The 30-40-50 and 60-ft. spans have the girders 7 ft. apart on

centers, and all of them except the 60-ft. girders have bearings with $\frac{3}{4}$-in. sole plates and $\frac{3}{4}$-in. bed plates, the former having at one end round holes and at the other end slotted holes for $1\frac{1}{4}$-in. rag bolts 12 in. long. All spans except the 100-ft. spans have zigzag single-angle lateral diagonals in the plane of the top flanges only, and all, including the 100-ft. spans, have vertical transverse sway-brace frames, made with horizontal and diagonal angles. In the 30-ft. spans these occur at alternate panel points; in all other spans, at every panel point. The vertical web-stiffeners are pairs of angles crimped except at the ends and at web-splice points, and are arranged to divide the web into panels decreasing in length but slightly from the center of the girder towards the ends, except at the extremities where they are very short, so that both angles are directly over the bearing plate. The 30-ft. girders are made with 48 $\times$ $\frac{3}{8}$-in. web plates and 6 $\times$ 6 $\times$ $\frac{1}{2}$-in. flange angles, both full length, and one 14 $\times$ $\frac{9}{16}$ $\times$ 20-ft. 9-in. cover plates on each flange.

The 40-ft. girder has a 60 $\times$ $\frac{3}{8}$-in. web without splices, and a T-shape bottom flange made with two 6 $\times$ 6 $\times$ $\frac{7}{16}$-in. angles and three 14 $\times$ $\frac{3}{8}$-in. cover plates from about 15 to 27 ft. long. The H-shape top flange is made with two 4 $\times$ 4 $\times$ $\frac{7}{16}$-in., two 6 $\times$ 4 $\times$ $\frac{7}{16}$-in. angles, and one 14 $\times$ $\frac{3}{8}$-in. full-length cover plate, and two 6 $\times$ $\frac{3}{8}$-in. side plates 15 ft. 7 in. long. The lower angles in the top flange are cut nearly 1 ft. short at each end, and the horizontal flanges of the top and bottom angles are fitted to bearing on short sections of the web-stiffener angles. The lower part of the end panel is stiffened to make a vertical post over the center of the bearing by two pairs of angles, back to back, riveted to the web.

The 50-ft. girder is like the 40-ft. girder, except for dimensions and materials. The 72-in. web is made in two pieces, spliced in the center with two 13 $\times$ $\frac{9}{16}$-in. cover plates with four vertical rows of rivets, and the top flange is made with four 6 $\times$ 4-in. angles with the 4-in. flange of the upper angles and the 6-in. flanges of the lower angles horizontal.

In the 60-ft. girder the principal differences from the 50-ft. girder, aside from the dimensions and materials, are that the web is 78 in. deep, the lower flange angles are 6 $\times$ 6 in., and that the sole plates do not project beyond the edges of the flange angles and are seated on the between guide strips on cast-steel pedestals to which they are secured by four 1-in. bolts engaging round holes at the fixed and slotted holes at the ex-

pansion end. The pedestal is planed top and bottom and secured to the masonry with four $1\frac{1}{4}$-in. rag bolts 12 in. long. It is 6 in. high with three longitudinal and four transverse vertical webs, and its base is extended to $17\frac{1}{2} \times 29$ in.

The 70-ft. girder has a $100 \times \frac{7}{16}$-in. web with two splices in each of which the upper edges of the plates are made $\frac{1}{8}$ in. longer than the lower edges so as to make an obtuse angle at the joint and from a camber into the girder. There are both top and bottom lateral diagonals in both 60 and 70-ft. spans, and in the latter the top diagonals are each made with a pair of 3×3-in. angles riveted together, back to back, with their horizontal flanges up.

The 80-ft. girders are 105 in. deep with four web splices, and each web plate $\frac{1}{8}$ in. longer on top than on the bottom edge. The web joints are spliced with pairs of vertical $16 \times \frac{3}{8}$-in. plates between the flanges, and by pairs of horizontal $8 \times \frac{3}{8}$-in. plates between the upper and lower angles of the top flange. Triple-web shoes are riveted to the ends of the lower flanges, and are connected by $4\frac{1}{2}$-in. pins to interlocking riveted pedestals seated at the expansion end on 6-in. segmental rollers and bed plates, and at the fixed end on bolsters of an equal height made with pairs of longitudinal channels, back to back, with top and bottom plates, the top plate serving as the base plate of the pedestal and being shop-riveted integral with both pedestal and bolster.

The 90-ft. girders have 110-in. webs with five splices, and have 8×8-in. bottom full-length bottom flange angles. The pin bearings are 88 ft. apart on centers, and the bottom flanges are $21\frac{1}{2}$ in. clear of the abutment masonry seats.

The 100-ft. girders have webs 112 in. deep, and each flange angle is made in two pieces with the joint from 12 to 22 ft. from the ends of the girder. The joints of the same pair of angles are on opposite ends of the girder and on opposite sides of the web, and are spliced by plates bent to make cover angles for the bottom flange and by extensions of the side cover plates for the top flange angles. The shop rivets in the flanges of the 90 and 100-ft. girders are 1 in. in diameter, all other rivets are $\frac{7}{8}$-in. diameter. The estimated weights of the complete spans are 30 ft., 14,900 lb.; 40 ft., 23,800 lb.; 50 ft., 34,300 lb.; 60 ft., 47,500 lb.; 70 ft., 68,000 lb.; 80 ft., 87,800 lb.; 90 ft., 113,200 lb.; 100 ft., 137,800 lb.

A solid-floor type of bridge has been developed for short

span, which in the case of a two-span double-track bridge at Elkhorn, has six lines of deck plate girders 30 ft. long, 30 in. deep, and 4 ft. 8 in. apart on centers, which are seated on cast pedestals and connected by ordinary sway-brace frames at the ends only. The girders meet on common pedestals on the center pier, and their ends are field-riveted together through the vertical web-stiffener angles, making continuous girders. A solid deck of 12-in. channels with horizontal webs is riveted across the top flanges and overhangs the side girders $3\frac{1}{2}$ ft. The ends of the channels are web-connected to 15-in. longitudinal fascia I-beams, and the deck is covered with a solid bed of concrete on which the ballast is laid.

A short-span double-track through bridge with a very shallow floor has three 35-ft. girders, 48 in. deep and 14 ft. apart

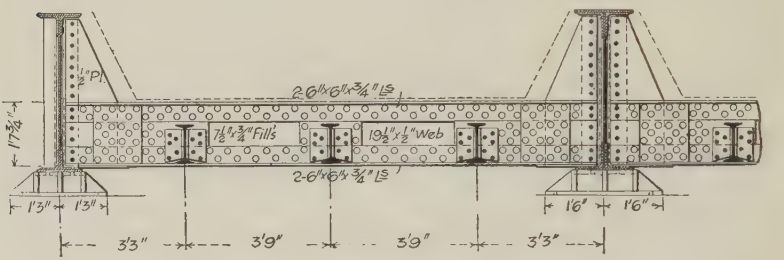


Fig. 66.

on centers, made without splices. Both flanges are alike, and are made with two 6 × 6 × $\frac{1}{2}$-in. angles and two 14 × $\frac{1}{2}$-in. cover plates. The girders are bolted through round and slotted holes to cast pedestals similar to those described for short-span standard-plate deck girders, except that their bases are extended to 36 in. wide and 24 in. long for the center girder. The floorbeams are 5 ft. 5 in. apart with the dimensions and connections shown in the half cross-section of the span. The end sections of the webs form kneebraces field-riveted to the single web-stiffener angles of the main girders which both flange angles on the same side of the floorbeam are cut to clear at both ends.

The three lines of I-beam stringers, 45 in. apart, for each track, are extended beyond the end floorbeams by brackets carrying one track tie over each abutment. The 8 × 8-in. ties are 12 ft. long and 11 in. apart on centers, and every third one is hook-bolted to the outside stringers. The 12 × 12-in. guard

timbers are dapped over the ties and bolted to every third one. The lateral diagonals each cross one floorbeam but do not cross the center girder; they are single 6-in. channels with horizontal webs field-riveted to connection plates engaging the bottom flanges of both girders and floorbeams. In each panel one diagonal has its flanges turned down and the other has its flanges turned up and cut to clear at the intersection where both are riveted with fillers to a connection plate field-riveted across the bottom flange of the floorbeam, but not to the stringer. The center girder has a $\frac{5}{8}$-in. web and a maximum flange section made with two $8 \times 8 \times \frac{3}{4}$-in. angles and three $18 \times \frac{9}{16}$-in. cover plates.

A deep-floor double-track through bridge has three girders 60 in. deep and 35 ft. 10 in. long with floorbeams 31 in. deep and 10 ft. 10 in. apart which have their webs made in single pieces with separate kneebrace plates riveted to their top flanges and to the web-stiffener angles of the main girders. The stringers are 18-in. 65-lb. I-beams, and the lateral diagonals are single $3\frac{1}{2} \times 4\frac{1}{2}$-in. angles.

In the 50-ft. span the girders are 60 in. deep with a full-length flange angle and two splices in each web. The side girders are made with 6×6-in. flange angles and 16-in. cover plates and a $\frac{3}{8}$-in. web. The center girder has a $\frac{1}{2}$-in. web, and each flange is

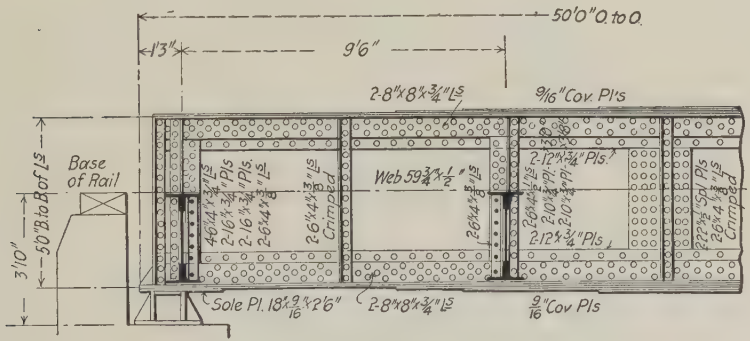


Fig. 67.

made with two $8 \times 8 \times \frac{3}{4}$-in. angles, two $13 \times \frac{3}{4}$-in. side plates, and four $18 \times \frac{9}{16}$-in. cover plates. The $\frac{1}{2}$-in. web is spliced with a pair of $22 \times \frac{1}{2}$-in. cover plates with six vertical rows of rivets. Where these spans replaced old single-track deep-floor through spans of the same length, the girders of the latter were

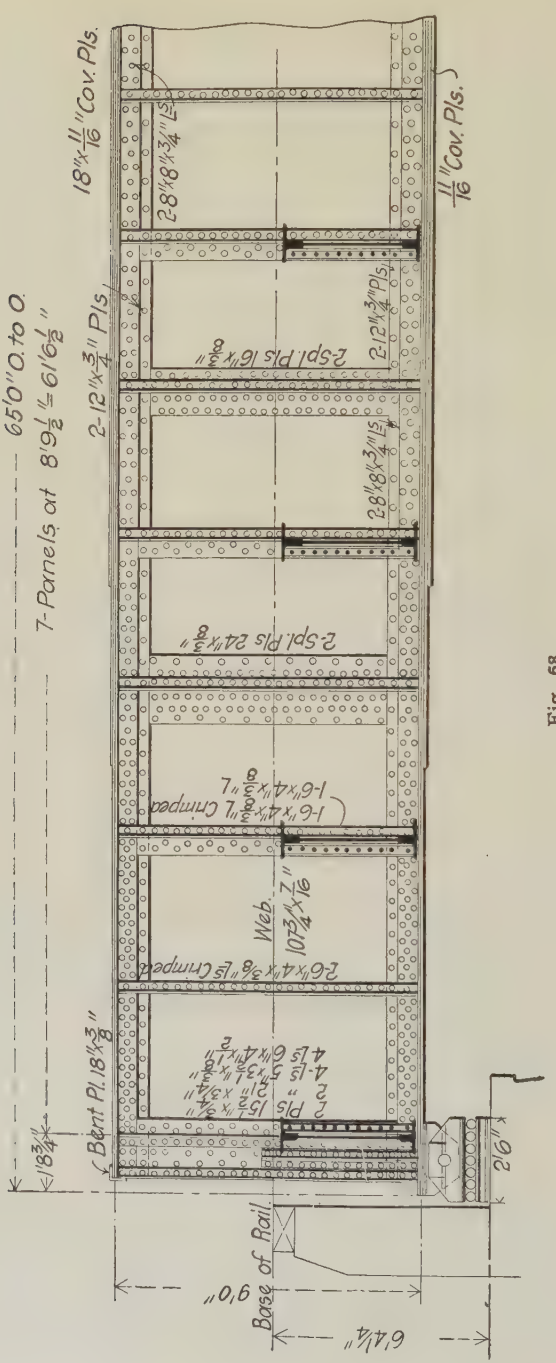

65'0" O. to O.
7-Panels at 8'9½" = 61'6½"
2-12"x¾" Pls
18"x11/16" Cov. Pls.
11/16" Cov. Pls.
2-8"x8"x¾" Ls
2-12"x¾" Pls
2-Spl. Pls 16"x⅜"
2-8"x8"x¾" Ls
2-Spl. Pls 24"x⅜"
1-6"x4"x⅜" L Crimped
1-6"x4"x⅜" L
Web. 10¾"x7/16"
2-6"x4"x⅜" Ls Crimped
2 Pls 15½"x¾"
2 " 21" x¾"
4-Ls 5"x3½"x⅜"
4 Ls 6"x4"x7/16"
Bent Pl. 18"x⅜"
18¾"
21⅝"
Base of Rail
9'0"
6'4½"

Fig. 68.

separated to form the outside girders of the new spans, and new heavy center girders were set between them and an additional floor system provided.

Double-track through bridges of 65-ft. span are made with two girders 108 in. deep spaced 30 ft. apart on centers. Each

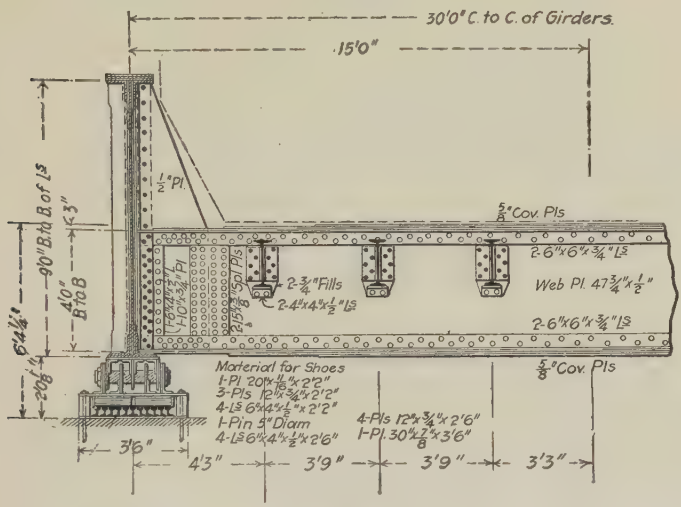


Fig. 69.

web is cambered ¼ in. and made in five $\frac{7}{16}$-in. pieces. The two center splices are made with a pair of 16 × ⅜-in. cover plates and 98⅞-in. shop-driven rivets in four vertical rows. The end splices are made with two 24 × ⅜-in. cover plates and 128 rivets, including those through the flanges, in six vertical rows. The flanges are made without splices and have a maximum section of two 8 × 8 × ¾-in. angles, two 12 × ¾-in. side plates, and three 18 × $\frac{11}{16}$-in. cover plates. Over the bearings the webs are reinforced by cover plates and by four full-length vertical web-stiffener angles and four half length. The girders have pin bearings with shoes, pedestals, roller nests, and bolsters. The floorbeams are 48 in. deep with the end sections of their webs projecting above their top flanges to make gusset kneebrace plates. There are three lines of 18-in. I-beam stringers for each track. The centers of the floorbeams are braced by a longitudinal line of 4 × 4 × ½-in. angles in the plane of the lower flanges, and the panels on each side of these struts are X-braced by 4 × 3½-in. angles with the vertical flanges of the angles in the same panel turned in opposite directions to clear

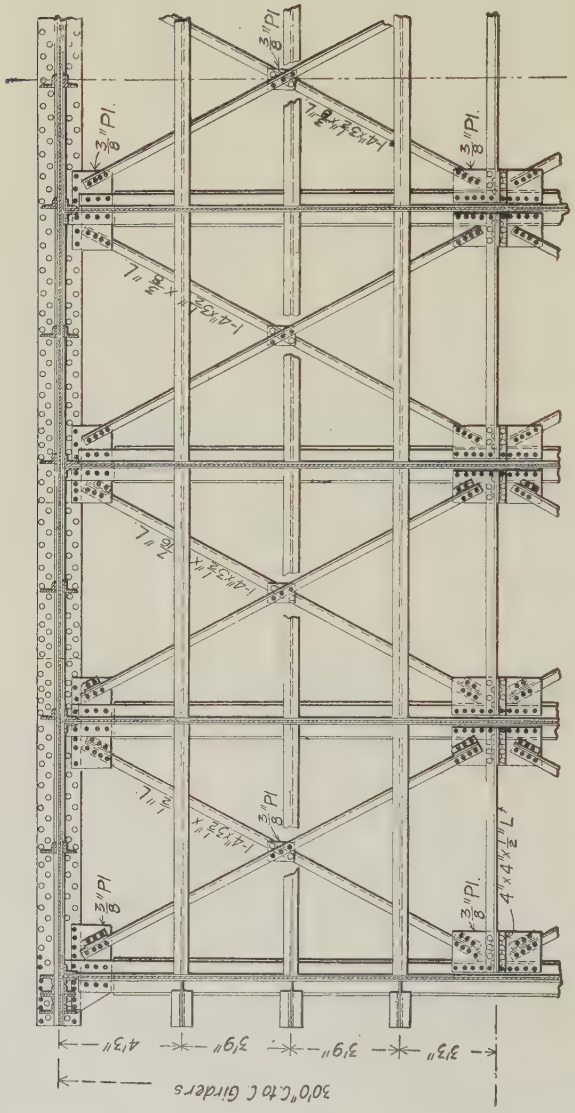

3/8" Pl.
3/8" Pl.
3/8" Pl.
3/8" Pl.
3/8" Pl.
1-4"x3½"x⅜" L
1-4"x3½"x⅜" L
1-4"x3½"x⅜" L
1-4"x3½"x⅜" L
1-4"x3½"x⅜" L
4"x4"x½" L
3'3"
3'9"
3'9"
4'3"
30'0" C. to C. Girders

Fig. 70.

at intersections where, and at the ends, they are field-riveted to $\frac{3}{8}$-in. connection plates. For single-track bridges the same

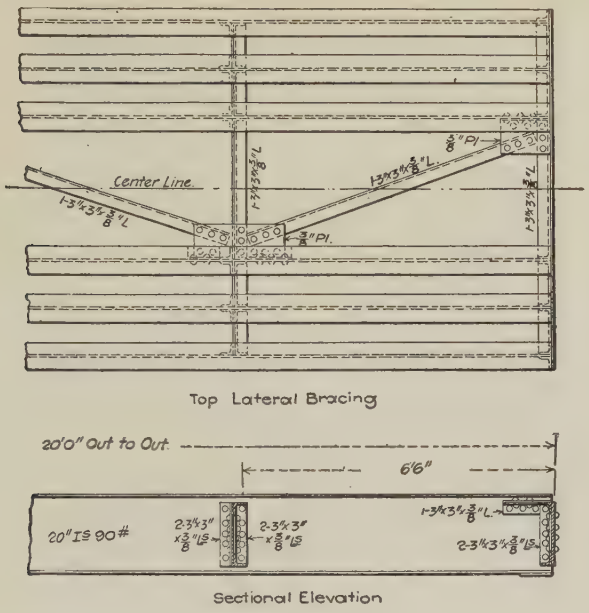


Fig. 71.

girders are used with temporary floorbeams 30 in. deep and 16 ft. long. In these cases the inclined edges of the kneebrace

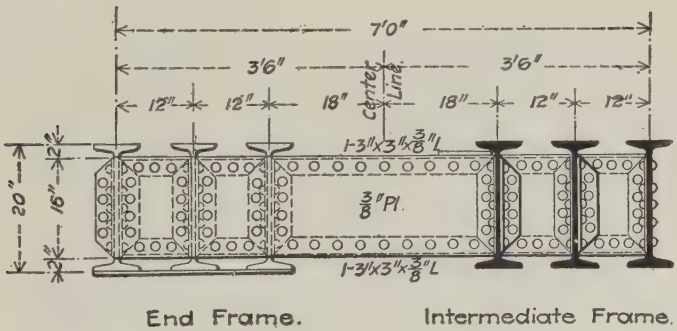


Fig. 72.

plates have flange angles which are bent and have their upper ends vertical to provide sufficient clearance. When the bridges

are double-tracked the floorbeams are removed, the girders separated and new floorbeams inserted as previously described.

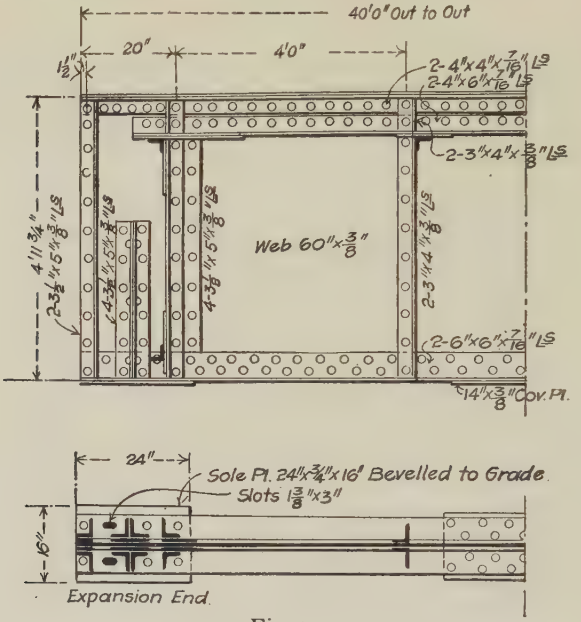


Fig. 73.

Single-track 95-ft. through bridges correspond essentially to the 65-ft. spans except in dimensions and materials and in that

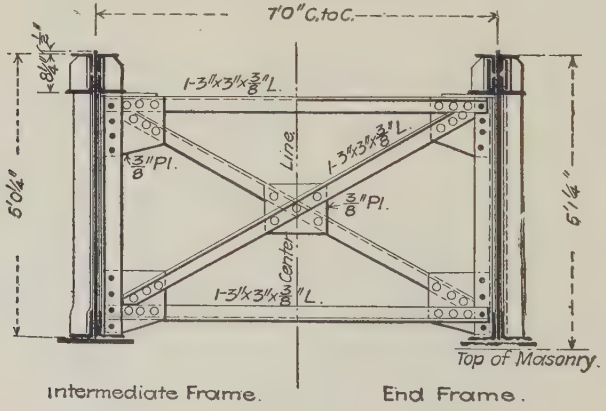


Fig. 74.

the flange angles and full-length cover plates are spliced. The top and bottom flange joints are staggered on opposite sides of

the web, and are opposite each other on the same side of the web. They are made with cover angles and with extensions

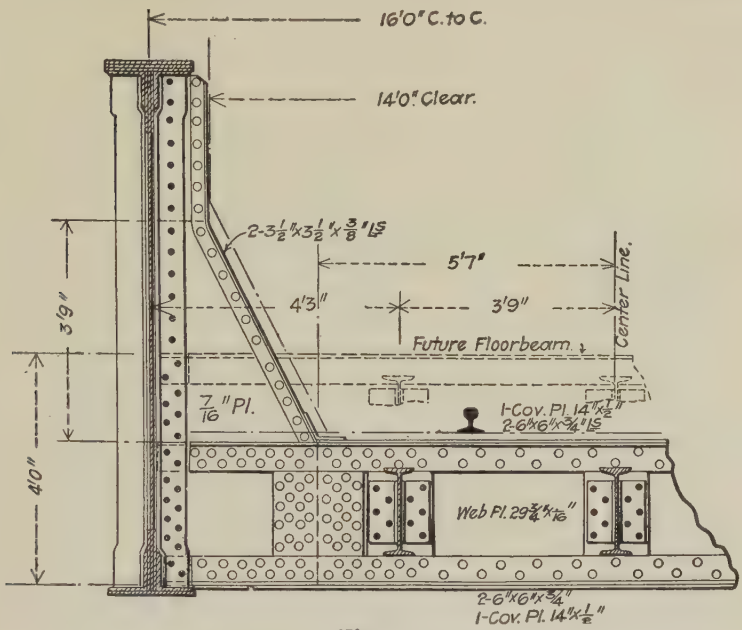


Fig. 75.

of the cover plate. The cover-plate joints are spliced by the extensions of the ends of the outside cover plates.

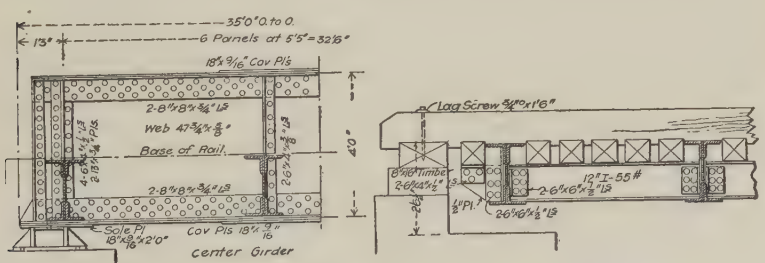


Fig. 76.

Boston and Maine Railroad.

Details of typical structures on the Boston and Maine Railroad are given, and some important practical considerations presented with a valuable discussion and conclusions by bridge engineer J. P. Snow. It was intended to publish these data in Vol. II of "Types and Details of Bridge Construction," but as

they related especially to railroad standards they have been reserved for the present volume, of which they are considered to form one of the most valuable portions. Mr. Snow's original communication under date of July 23, 1904, slightly modified

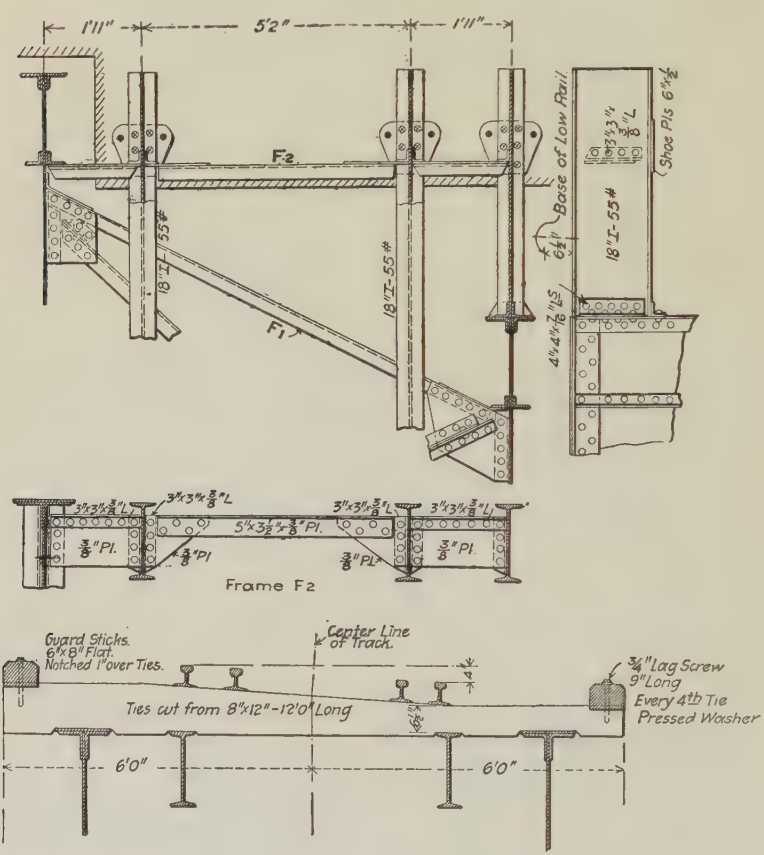


Fig. 77.

to conform to the few developments between that time and 1906, is, in part, as follows:

"I am sending you three prints covering a deck plate-girder bridge built in 1903 by the Boston and Maine Railroad. It is on a grade, a curve and skew location. We build all of our deck plate girders on this pattern. The girders are 9 ft. on centers with floorbeams making about 10-ft. panels. Stringers are 5 ft. on centers with tops flush with tops of girders. Our standard ties are 12 ft. long with spacing timbers at their ends.

"We formerly built the floorbeams lower so that the stringers rested on their tops similarly to the end beams in the present instance. We had so much trouble from twisted stringers and imperfect workmanship, however, that we are now building them so that the stringers can be headed into the beams. These bridges cost only about $5.00 per running foot more than plain deck girders properly braced. The floor makes a large part of the lateral and vertical bracing, so that the excess of cost due to the floor is not so much as might appear at first sight. The ties may be smaller than are required where there are no floorbeams and stringers, and the girders are spaced wide apart, and their serviceable life is a year or two longer than where they are depended upon to carry moment.

"On the whole it seems to me to be money well invested to build this type of deck bridge, and the Boston and Maine have so built them exclusively for the past 15 years, although so high an authority as Mr. J. A. L. Waddell says in 'Trans. of Am. Soc. C. E.,' Vol. 26, page 273, that they are unnecessarily expensive, 'as would be discovered by the general manager of any line of road who would try the experiment of building such structures.'

"These spans, if not more than 65 ft. long, can be readily shipped riveted up complete and put in place economically and without so much risk of injury as is involved in handling single girders.

"The drawings show our method of elevating the rail for curvature and the way that we square up the ends of skew bridges. We invariably do this at all skew bridges, long or short, wood or iron. It adds something to the cost of the masonry, but if not done we get woeful complaints from the road department on account of the difficulty of maintaining a good surface over the approach.

"The drawings also show our standard parapet which allows the first grade tie to be placed at regular spacing from the last bridge tie. On many roads this feature is accomplished by leaving the parapet low enough to receive a tie directly on the stone. This is in effect topping out the abutment with wood. When so constructed the ballast lies against the tie, and it and the dampness absorbed from the stone combine to decay the tie much earlier than is the case with the regular bridge ties. This tie is a sawed tie like those on the bridge, so that the sectionmen will have none on hand with which to renew it

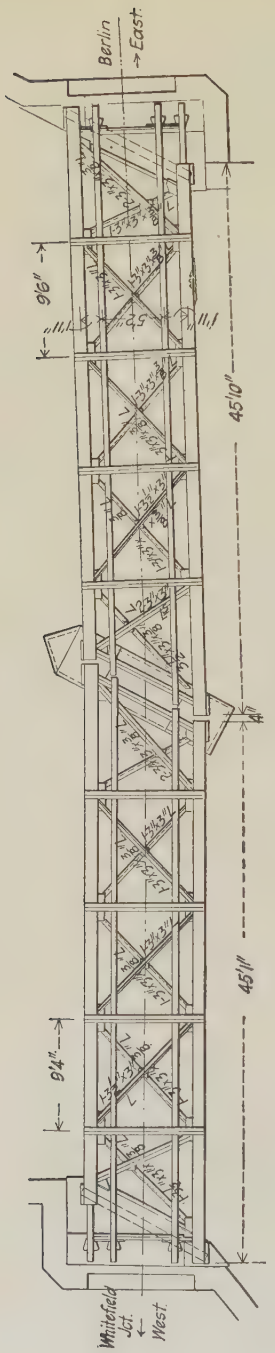


Fig. 78.

when decayed, and it will call loudly for renewal before the bridge crew will be sent to renew a single tie. The net result is that this parapet tie may often be found in very poor condition. The scheme shown on the plan seems to me much better.

"The plans show a comparatively thin wall plate. Ordinarily it is better, on a pier at least, to use a chair 12 or more inches high. I prefer cast iron for wall plates of all kinds because they resist corrosion so much better than rolled material, and if high enough they keep the girders out of the dirt and also allow an inspector to get on all sides of the structure."

Bridge A-263, of the White Mountains Division of the Boston and Maine Railroad, over Berlin Mills Canal, although of special construction to provide for the grade, curve and skew, is essentially typical of the standard single-track deck plate-girder spans of that road. There are two spans of 45 ft. 10 in., and 45 ft. 1 in., with a skew of 5 ft. 1½ in. for the middle pier and one abutment, and 4 ft. 9 in. for the other pier.

In the center of the bridge the track is on a 4-degree curve which intersects the longitudinal axis of the bridge near the middle of each span; beyond these points the track is on tangents so that its center is about 3 in. on one side of the axis at each end, and about 3 in. on the opposite side in the center of the bridge.

The girder web is 60 in. deep and ⅜ in. thick throughout, with two shop splices, each made with a pair of cover

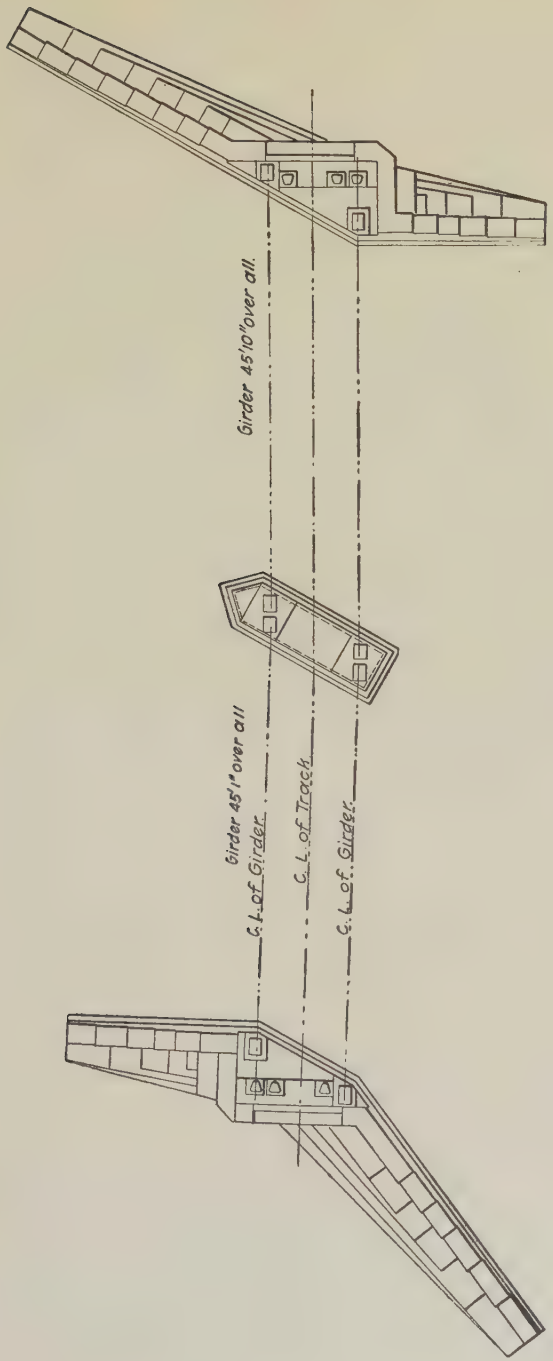

Girder 45'10"over all.
Girder 45'1"over all
C. L. of Girder.
C. L. of Track.
C. L. of Girder.

Fig. 79.

plates, and two vertical rows of rivets. The flanges are made without splices, and their maximum section consists of two $6 \times 6 \times \frac{1}{2}$-in. angles, and one $15 \times 1\frac{9}{16}$-in. cover plate. The end bearings are made with notched sole plates and cast bed plates similar to those shown in the Hook's Crossing Bridge, except that they are extended unsymmetrically on the outsides of the girders, where they are made thinner and receive the anchor bolts directly instead of through the guide lugs.

The floorbeams are 28 in. deep with 3×3-in. flange angles, the upper ones being reversed so that their vertical legs are up, and the horizontal legs are riveted across the top flanges of the girders. The upper corners of the webs are cut off to clear the girder flanges, and the ends are web-connected to the girders through two-short vertical angles at each end. The lower flanges rest on long horizontal shelf angles, which also serve as connection angles for the horizontal lateral plates, receiving the single lateral diagonal angles which have their flanges turned in opposite directions in the same panel and clear the stringers. The end floorbeams are oblique, and are set lower than the intermediate ones so as to support the stringers across their top flanges to which they are braced by deep vertical transverse plates with one edge field-riveted to the stringer web, and the opposite edge riveted to the girder web.

The track ties are supported on the top flanges of both stringers and girders, and all of these terminate at each end of the bridge in the same line perpendicular to the bridge axis so that the last bridge tie is supported wholly on the steel work, and the first shore tie is supported wholly on the abutment. To secure this condition, the short girder at each end of the bridge is extended about 5 ft. by an 18-in. I-beam riveted to the end, and supported at the shore end on a cast-iron pedestal like those of the stringers, and similar to those illustrated for Hook's Crossing Bridge.

At each end of the bridge the extremities of both girders are connected by lattice girders nearly parallel with the skew, with bent plate web connections. The ends of the stringers and the girders are connected by transverse horizontal braces, made with angles and vertical diaphragm plates. The abutments are special with wide triangular copings and long wing walls oblique to each other. The pier is rectangular in plan with the upstream end pointed, and it has seats for the girders only, the stringers being supported on the end floorbeams there and can-

tilevering beyond them to meet on the axis of the pier with a clearance of 2 in.

Hook's Crossing Bridge on the eastern division of the Boston and Maine Railroad, about half a mile from Salisbury, Mass., has

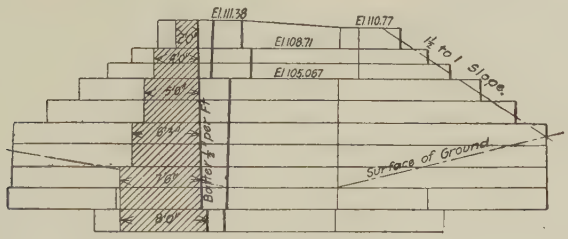


Fig. 80.

one double-track through skew-plate girder span of 56 ft. 9½ in., over all which is skewed 52 degrees or 20 ft. 3 in. The three girders are 13 ft. apart on centers, and are 66 in. deep with

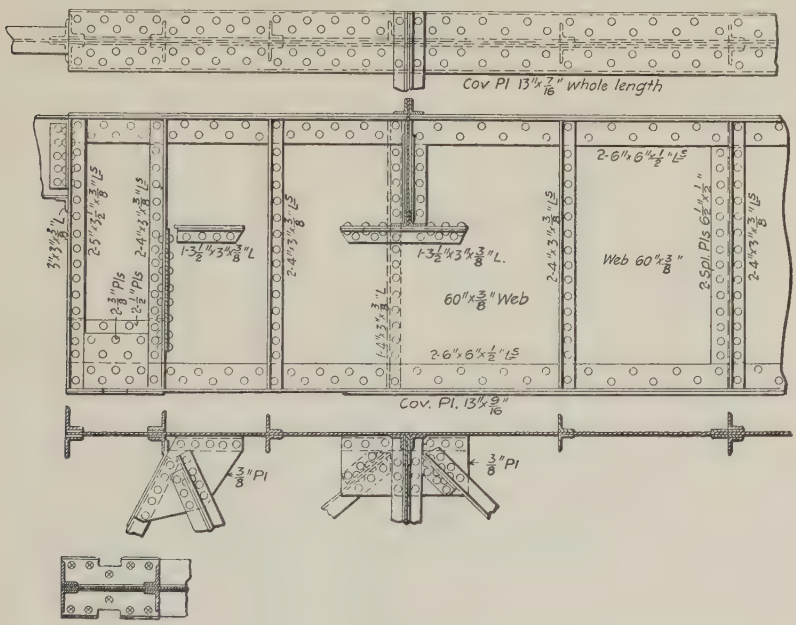


Fig. 81.

their $\frac{3}{8}$ and $\frac{7}{16}$-in. webs stiffened by pairs of 3 × 3-in. vertical crimped angles from 21 in. apart at the end to 38 in. apart in the middle panels. The webs are reinforced to a thickness of

$2\frac{7}{16}$ in. for a height of 12 in. from the lower flange in the end panels, and the bearings under these reinforcements are made

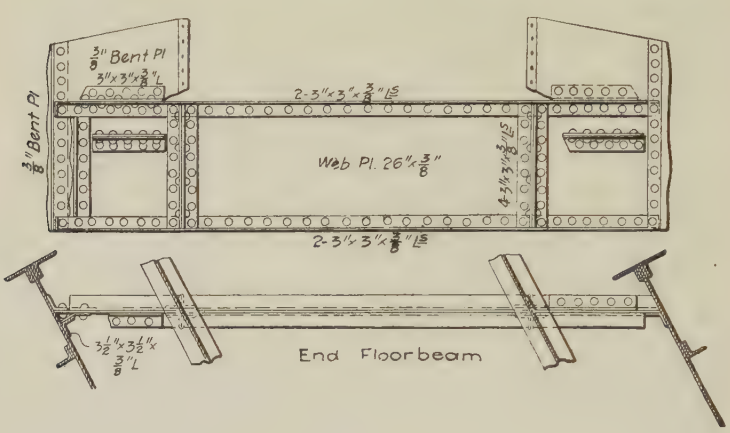


Fig 82.

with planed sole plates projecting $1\frac{1}{8}$ in. beyond the edges of the flange angles and notched to engage lugs on the cast bed plates.

The girders are connected by floorbeams about $12\frac{1}{2}$ ft. apart

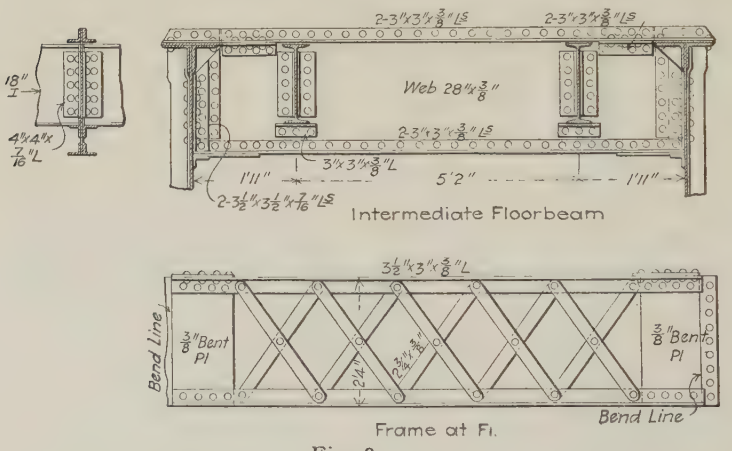


Fig. 83.

for each track, but which are staggered about $2\frac{1}{4}$ ft. on opposite sides of the center girder. The floorbeams are only 23 in. deep with their top and bottom flanges made with different size angles and reinforced by cover plates of different widths. They

are kneebraced to the girder webs by separate gusset plates without flange angles on their inclined edges. Horizontal connection plates are field-riveted to the lower flanges of the floorbeams and the girders, and receive the lateral angles in every panel, clear of the stringers.

There are two 15-in. 60-lb. stringers under each rail, and one 15-in. 50-lb. I-beam on each side of each track, making twelve lines on the bridge. They are web-connected to the floorbeams, and are all of the same length in each end panel, thus projecting far over the face of the abutment on one side, on account of the skew, to provide for the support of the ties entirely on the stringers until they are beyond the end of the bridge and are set entirely on shore. The shore ends of the stringers are seated on cast-iron pedestals, and are braced by horizontal transverse struts to the end of the long girder. Shelf angles are riveted to the floorbeams to receive the lower flanges of the stringers, and the floorbeam flange angles are cut at the ends on opposite sides of the webs to clear the girder web-stiffener angles and allow them to be revolved into place in erection.

The floorbeam connections are each made with one short and one long vertical angle shop-riveted to the floorbeam web; both angles are field-riveted to the girder web, and one of them is nearly the full height of the web, serving as a regular web stiffener, but cut to clear the top flange of the girder, so as to insure easy assembling in erection. The abutments for this bridge are similar to those illustrated in the description of the Berlin Mills Canal Bridge, but have faces about 66 ft. long parallel with the street, and wing walls about 26 ft. long perpendicular to the tracks. The rear of each abutment is offset horizontally to provide separate seats for the two tracks, and requires wide platforms on account of the great angle of the skew. The coping is made special so that the first shore tie rests on ballast instead of being seated on the masonry.

Referring to this bridge, Mr. Snow writes:

"I believe that the angle-iron uprights are more than sufficient to hold the top flange from side deflection without any gussets at all, but we put them in for fashion's sake. If these gussets or brackets are made as wide as is often recommended they cut the tie spacer timbers badly, and are in the way of snow-plow wings.

"This bridge shows the side stringers that we invariably use on all main track bridges, wood or iron. Our ties are 12 ft. long,

and any other reasonable arrangement of the stringers necessitates too much overhang of the ties. Our bridge carpenters invariably desire a support directly under the rail; and it seems reasonable, that the ties should be called upon to do only their legitimate duty of holding the rails to line and surface. If called upon to assist the floorbeam and carrying moment, which is what happens when the stringers are spaced more than 5 ft. on centers, the design is certainly faulty; for the rapidly decaying tie is depended upon to do what the iron work ought to do.

"It is claimed by some engineers that the metal used in side stringers ought to be distributed where it will add to the strength of the bridge, but a bridge has other functions than mere strength. One of these functions is to properly support the standard length of tie adopted by the road on which the bridge is located, and I know of no way to do this for a 12-ft. tie except by using side stringers in through bridges.

"The reinforcement of the flange rivets over the bearings shown on both these sets of drawings is preferred by us to schemes depending on the contact of stiffener angles on the horizontal flanges of the bottom angles. In riveted work dependence should be placed only on rivets. Machine fits on rolled surfaces are not to be counted upon as a sure thing. Many times I have found girders from our best shops with stiffeners which were supposed to make contact fits with the flange, off $\frac{1}{16}$ in.; the next stiffener perhaps would be $\frac{1}{16}$ in. too long, and the flange would be buckled down to that extent.

"The stone bolts of these bridges extend 4 in. into the masonry. This is much less than most engineers prescribe, but it is ample. All that is needed is something to keep the wall plate from working out of place. Two bolts for each plate are needed so that it cannot mill around. Very many old bridges have no dowel at all, or anything to confine them to place, but it is a very rare thing to find one working out of place. Often when broken in three or more pieces they show no tendency to get out of place. The holes for 1-in. bolts in the cast wall plate are cored $1\frac{1}{2}$ in. diam. With this size hole the stone can be drilled through the casting after it is in place, which often saves the erector a great deal of trouble; the dotted line on the floorbeam elevation shows the clearance line to which all of our structures are designed.

"I also send prints of a through plate-girder bridge built to carry the tracks over a street. This is a very common type with

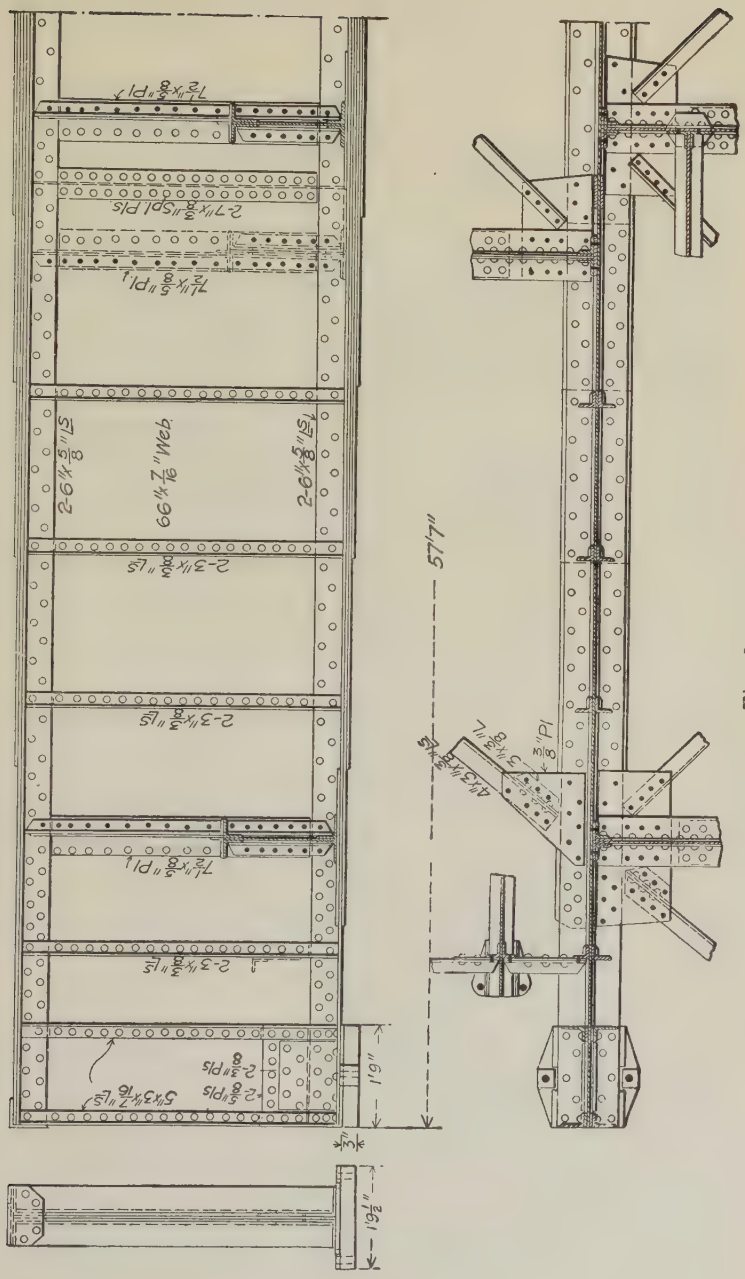

57'7"
19"
19½"
2-6"x⅝" Web
2-6"x⅝" Web
2-3"x3/8" L's
2-3"x3/8" L's
2-3"x3/8" L's
2-3"x3/8" L's
2-⅝" Pls
2-⅝" Pls
5"x3½"x⅞" L's
2-7"x⅜" Spl. Pls
7½"x⅝" Pl.
7½"x⅝" Pl.
7½"x⅝" Pl.
4"x3½"x⅜" L's
5"x3½"x⅜" L's
⅜" Pl.

Fig. 84.

us where highway grade crossings are eliminated. Short panels and shallow beams for stringers are used to save depth of floor. The bottom of stringers is kept above the bottom of the floor-

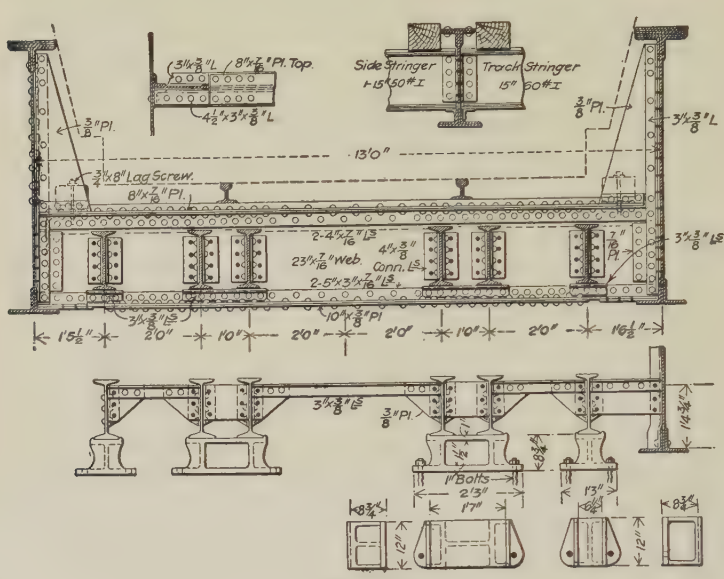


Fig. 85.

beam so that the bracing angles may be crossed without cutting them. This arrangement produces a floorbeam as shallow as can be used without awkwardness. We frequently countersink the rivets in the top plates of the floorbeams under the rails to gain a little depth. The space between base of rail and highest iron under it should be at least $1\frac{1}{4}$ in., or they will be in contact before the ties are worn out. I have known of many cases of serious interference of the rails and floorbeam flanges where less space than the above was allowed.

"It will be noticed that the connection of the floorbeams to the girders is such that there is no complete cut-off of the top flange as is frequently seen in through plate-girder designs. The connection angles are shop-riveted to the floorbeam, and as one of these angles is an upright of the girder care must be used in placing the rivets in the top flange so as to allow of easy erection.

"The knee gussets of ends of floorbeams are quite narrow. I do not think these knees of so much value as many designers appear to."

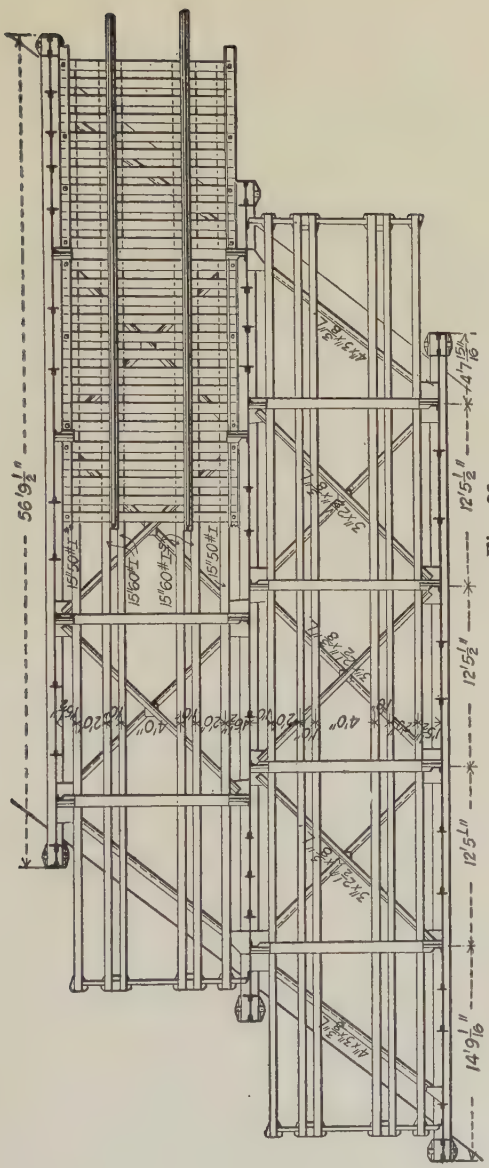


Fig. 86.

CHAPTER VIII.

RECENT TYPICAL SPANS.

The Pennsylvania Railroad.

THE longest plate-girder span on the Pennsylvania Railroad forms part of the track elevation at Market and Penn streets, in Chester, Pa., on the line of the Philadelphia, Baltimore and Washington Railroad. It consists of four pairs of girders 122 ft. long over all and about 9½ ft. deep, which are seated on masonry piers about 115 ft. apart in the clear. They are braced together with zigzag top and bottom lateral angles and transverse frames between each pair of girders, so as to make essentially four separate parallel spans, each carrying a single track with the cross-ties resting directly on the top flanges of the girders 8 ft. apart on centers. The adjacent girders of successive pairs are 5 ft. apart on centers and are connected by horizontal transverse angles 9 ft. apart, which serve to space them, but cannot transmit any stress from one track to the next.

The girders are of the simplest possible construction, designed with special attention to uniformity and duplication of members. The web is 114 in. deep and ⅝ in. thick throughout, and is made in two-panel lengths of 9 ft. 9½ in. except at the ends, where the pieces are 11 ft. 7½ in. long, and are reinforced over the bearings by a pair of 8-in. vertical angles. All web splices are made of a pair of 14 × ⅝-in. cover plates with four vertical rows of rivets having the same number at every splice. At intermediate panel points 4 ft. 11 in. apart, the web is stiffened by a pair of 7 × 3½-in. vertical angles with fillers. The web is cut ½ in. short of the actual length and depth so as to insure clearance at the splices and with the flange cover plates.

The T-shaped top and bottom flanges have a maximum cross-section consisting of two 8 × 8 × ⅝-in. angles and five 20 × ⅝-in. cover plates. Both top and bottom flanges have the same gross area and are made with exactly the same spacing so as to be duplicates except for the connections to the end shoes. The

angles are made with two rows of rivets in each flange, and are remarkable in that they are rolled full length without splices. Each bar weighs about 4,000 lb., and is one of the longest heavy angles ever rolled. The cover plates are from 32 to 122 ft. long, and are made so that the projecting ends of the outside plates splice the joints in the first two plates, each of which is made in two or three pieces. The width of the flanges is maintained at the ends of the girders by a $\frac{3}{8}$-in. cover plate riveted to the end web-stiffener angles.

The lateral angles are alike in both top and bottom systems, except that the direction of the diagonals is reversed in each. They are uniformly $5 \times 3\frac{1}{2} \times \frac{3}{8}$ in., and are field-riveted at both ends to $\frac{3}{8}$-in. connection plates field-riveted to the flanges. Transverse frames are made with single horizontal and diagonal $3\frac{1}{2} \times 3\frac{1}{2}$ in. angles, shop-riveted together and field-riveted to the vertical web-stiffener angles.

Cast-steel shoes 119 ft. 3 in. apart on centers are secured to the lower flanges of the girders with eight $1\frac{1}{8}$-in. turned bolts in each. They have continuous semi-cylindrical bearings for 5-in. pins which are seated in corresponding bottom castings or pedestals like the shoes, except that their bases are extended longitudinally and transversely. The pins have solid collars 1 in. wide and $\frac{1}{4}$ in. thick, turned on each end, which engage recesses in the ends of the shoes and pedestals and, together with flanged rings secured by the nuts, lock the pedestal and shoe together.

At the fixed end the shoe has a height of 11 in. to the center of the pin, and has a planed base seated on a $1\frac{1}{8}$-in. planed bed plate with a 1-in. anchor bolt through each corner of shoe and plate. At the expansion end the planed lower surface of the pedestal has a longitudinal guide rib engaging slots in the nest of eight 4-in. rollers on which the pedestal is seated. The rollers move on 4-in. longitudinal strips riveted to a $\frac{7}{8}$-in. bed plate. The ends of the rollers have shouldered bearings in side strips and the nest is inclosed by Z-shaped longitudinal pieces bolted to each side of the bed plate so as to engage the edges of the pedestal base, with their top flanges. These are made of pairs of angles riveted together through their vertical flanges, and serve to exclude dust and rubbish, but can be easily removed for inspection and oiling. The bridge was built at the Pencoyd plant of the American Bridge Co., and weighed 234,358 lb. for each track.

The bridge was built of soft open-hearth steel, punched and

unreamed except for pieces more than $\frac{5}{8}$ in. thick. It was designed according to the standard specifications of the Pennsylvania Railroad Co., which at the time of its construction differed little from the revised requirements now in force. The latter provide for a dead load made up of the steel weight, the timber estimated at $4\frac{1}{2}$ lb. per foot B. M. and 160 lb. per linear foot of track for rails, splices, etc. The live load for each track is a 5,000-lb. train preceded by two coupled consolidation loco-

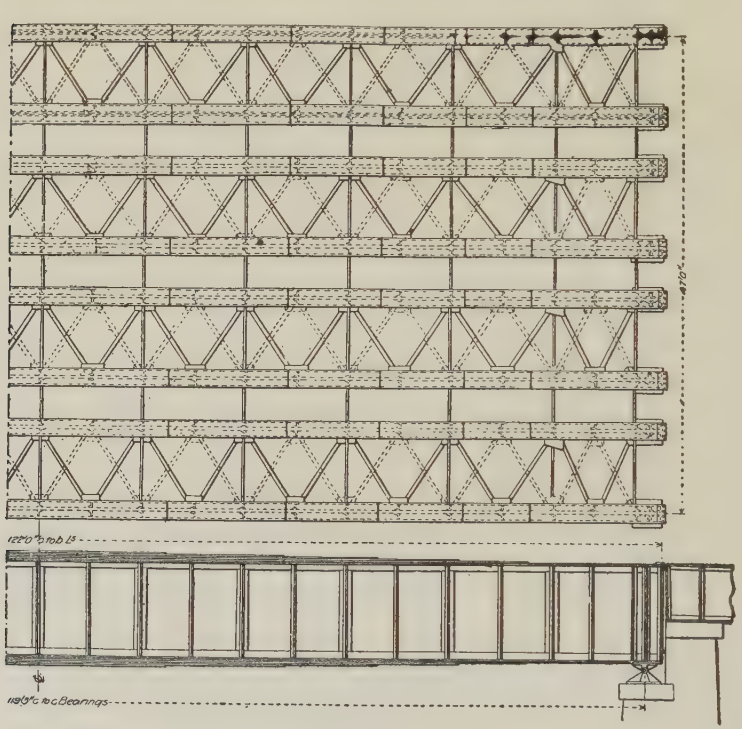


Fig. 87.

motives each weighing 374,000 lb. on a 107-ft. wheel base and having four pairs of drivers $5\frac{1}{2}$ ft. apart with 52,000 lb. on each axle. Wind pressure is assumed at 50 lb. per square foot on the vertical elevation of the unloaded girder, and 130 lb. on the same surface plus a train area 10 ft. high and $2\frac{1}{2}$ ft. above the base of rail, which is assumed to move across the bridge. Anchorage is provided for an assumed train load of 800 lb. per lineal foot Impact allowance recognizing both live and dead loads is provided by the formula, corrected stress,

$S = M(1 + k)$, where k is a coefficient calculated by the formula $k = 1 - 2R + R^2$. M is the calculated maximum stress, m is

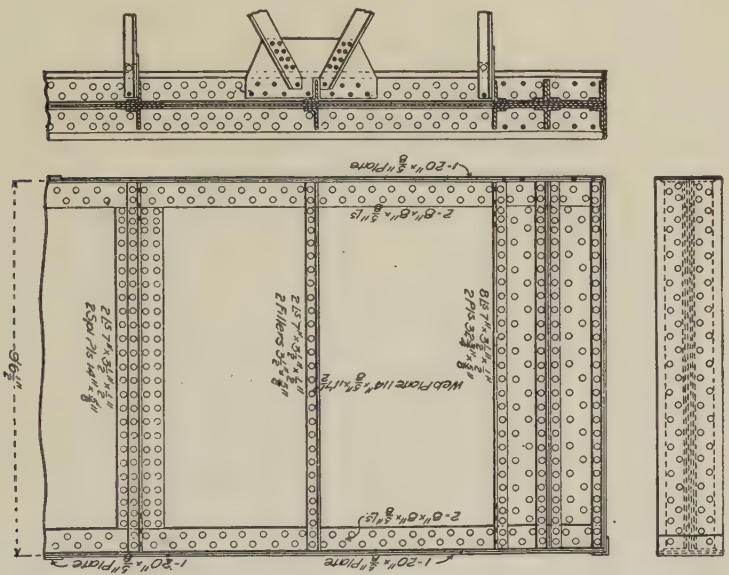


Fig. 88.

the calculated minimum stress, $R = M/m$. In order to insure the stability of bridges under increased live load, calculations

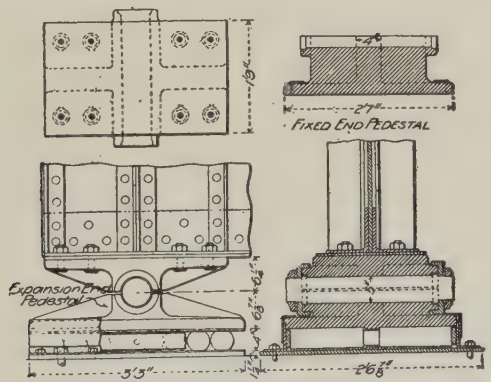


Fig. 89.

were made for a live load 100 per cent greater than that specified, and if under this assumption $S = M(1 + k)$ produced a unit stress more than twice as large as the allowable

working stress, the section was increased until that limit was not exceeded. The maximum working stress per sq. in. was not allowed to exceed 16,000 lb. for vertical loading or 20,000 lb. for combined vertical wind and momentum stress. Rivets were proportioned for bearing and shearing stresses of 24,000 lb. and 12,000 lb. per sq. in. respectively.

In calculating shearing and bearing stresses in the web rivets the whole of the shear, with its proper increment, acting on the side of the panel next the abutment was considered as transferred into the flange angles in a distance equal to the depth of the girder. The unsupported length of the compression flange was limited to twelve times its width. Rivets connecting the top flange angles to the web are proportioned for a 15-ton driver load distributing over three ties. The web is proportioned for a maximum shearing stress of 12,000 per sq. in. of gross cross-section.

Provision is made for expansion due to temperature variation of 150° Fahr. The expansion rollers are proportioned for a pressure per lin. in. not exceeding $1{,}000\sqrt{d} + 100\,d$, where d is the diameter of the roller in in. Bed plates are proportioned for a maximum load of 500 lb. per sq. in. on the masonry.

The bridge was designed and erected under the direction of the engineering department of the Pennsylvania Railroad.

New York, New Haven & Hartford Railroad.

Bridge No. 19 over Wicketequock Cove, near Stonington, Conn., on the Stonington branch of the Shore Line division of the New York, New Haven & Hartford Railroad, illustrates the principal features of this company's regular standard, double-track two-girder through structures of moderate span. It is 71 ft. 4 in. long over all, has a clear opening of 64 ft. 2 in., and carries the rail at an elevation of 4 ft. $7\frac{1}{2}$ in. above the lower flange of the girder. The web is made throughout with 96 × $\frac{5}{8}$-in. plates, with four splices like the one shown. The 8 × 8 × $\frac{3}{4}$-in. flange angles are made in single full-length pieces, and the maximum flange section has five 20 × $\frac{5}{8}$-in. cover plates, the shortest 31 ft. long. The distances apart of the vertical stiffeners vary up to a maximum of 5 ft. 9 in. at the center of the girder, and where the floorbeams are connected there is a web-stiffening angle on the outside only.

The floorbeams are 48 in. deep, with separate kneebraces field-riveted to their top flanges, and reaching to the top flanges of the girders. The stringers are 28 in. deep, and besides being web-connected to the floorbeams are seated on bracket angles with reinforced horizontal flanges. The bottom flanges of the stringers are field-riveted to the lateral angles, and the latter are field-riveted at the ends to horizontal connection plates engaging the webs of the floorbeams and of the main girders several inches clear of their lower flanges.

The shore ends of the stringers are in the same transverse lines as the ends of the main girders, and the extremities of their webs are reinforced by a pair of vertical web-stiffener angles 18 in. from the end which provide for the maximum shear over the flange bearing on a $9 \times \frac{3}{4} \times 18$-in. planed sole plate with a 12-in. bed plate having longitudinal guide ribs on both edges. The ends of the intermediate stringers are braced by a horizontal transverse strut like a light-lattice girder which is field-riveted to the vertical web-stiffener angles. The ends of the outside stringers are braced to the main girders by solid-web plate-girder-like struts making an oblique angle with the axis of the bridge so as to engage the girder stiffeners at the points of connection with the bottom lateral angles, about 28 in. from the ends of the girders. The girders have cast-steel shoes and pedestals similar to those described for deck girders on the same road in "The Engineering Record" of September 30, and their details, loading, materials, etc., correspond to those there described. This span weighs about 115 tons.

Bridge No. 6 of the East Providence branch of the Worcester division of the same railroad is a single $78\frac{1}{2}$-ft. span across the Providence division, and is characteristic of standard construction for a three-girder double-track skew through plate-girder span. The girders are $15\frac{1}{2}$ ft. apart, 9 ft. deep, and are divided into five 11 ft. 8-in. intermediate panels by the 32-in. floorbeams. The web of the center girder is of a uniform thickness of $\frac{9}{16}$ in., and is spliced in every panel with double cover plates and four vertical rows of rivets. At each lower corner it is reinforced by four $\frac{3}{8}$-in. plates riveted on over the end bearings to provide for the maximum shear. At each splice and at one or two points between the splices in each panel are a pair of vertical $6 \times 3\frac{1}{2}$-in. angles and fillers.

The maximum flange section consists of two unspliced $8 \times 8 \times \frac{3}{4}$-in. angles, and one full length and one 62 ft. 8-in. cover

plate each 21 in. wide and $\frac{5}{8}$ in. thick, and four 21 × $\frac{1}{2}$-in. cover plates. The $\frac{5}{8}$-in. plates are each made in two pieces with their joints arranged so that they are spliced by the projecting ends of the $\frac{1}{2}$-in. plates above them. The 8 × 8-in. angles have joints staggered in the two middle panels on opposite sides of the web, but opposite each other in the upper and lower flanges.

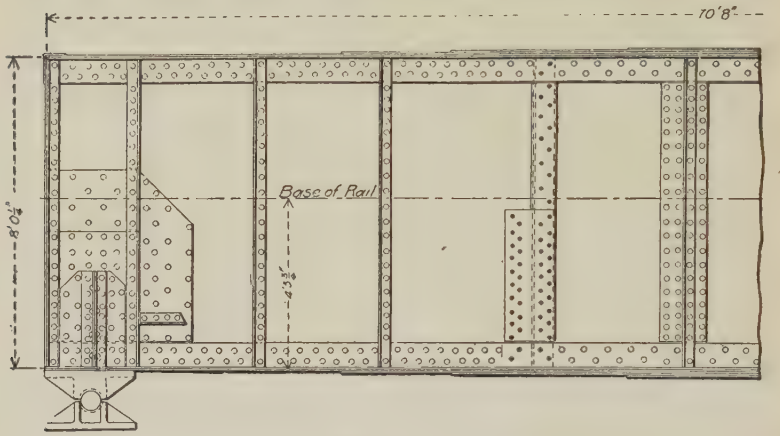
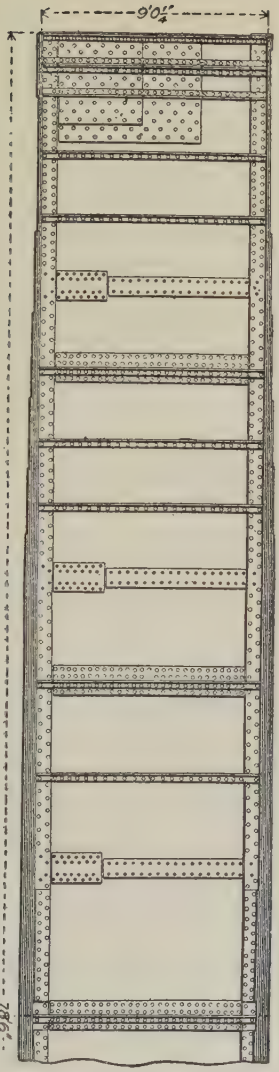
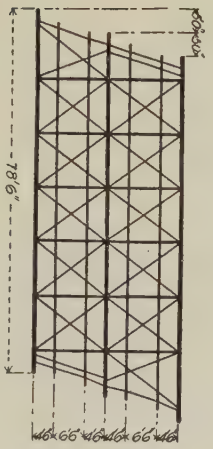


Fig. 90.

The angles are cut in lengths of 36 ft. 6 in., and 42 ft. 0 in., and their joints are spliced with pairs of sheared 8 × 8 × $\frac{3}{4}$-in. angles about 4 ft. 9 in. long. The outside girders are substantially like the center girder, except that the web is only $\frac{7}{16}$-in. thick, and has no reinforcement plates at the ends, and the maximum flange section has only three 21 × $\frac{1}{2}$-in. cover plates.

The floorbeams and their deep kneebraces are riveted to the girder webs with fillers which space their connection angles out to the faces of the vertical legs of the flange angles. Each panel between floorbeams is X-braced by single $3\frac{1}{2} \times 3\frac{1}{2}$ or 4 × 4-in. angles with their vertical flanges down, and the horizontal flanges riveted to the stringer flanges at intersections, and to connection plates engaging the bottom flanges of the girders and the floorbeams. Where the X-brace angles intersect, one of them is continuous, and the other is cut to clear it and spliced with a plate or with a plate and an angle. In the end panels care is taken to develop the full strength of the angles in the connection by the use of short connection angles riveted to the vertical flanges and to the connection plates.





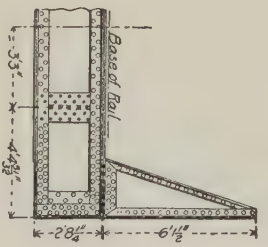


Fig. 91.

The braces at the shore ends of the stringers are like those illustrated for bridge No. 19. The bearings are modified from

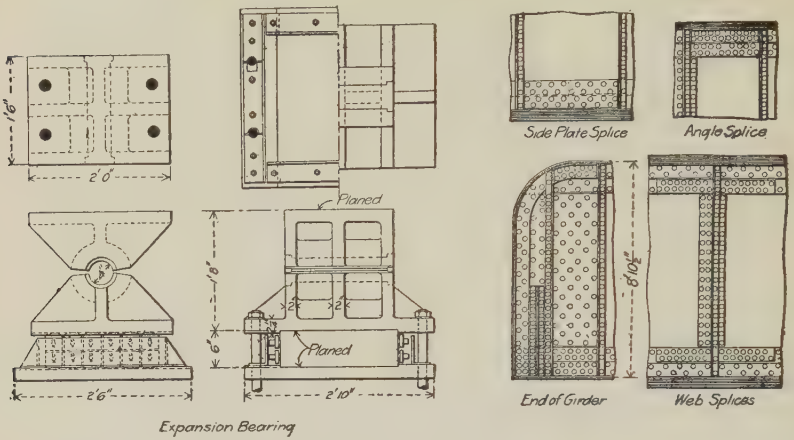


Fig. 92.

those already described in that the transverse webs of the shoe and pedestal are made much thinner and have a semi-cylindrical

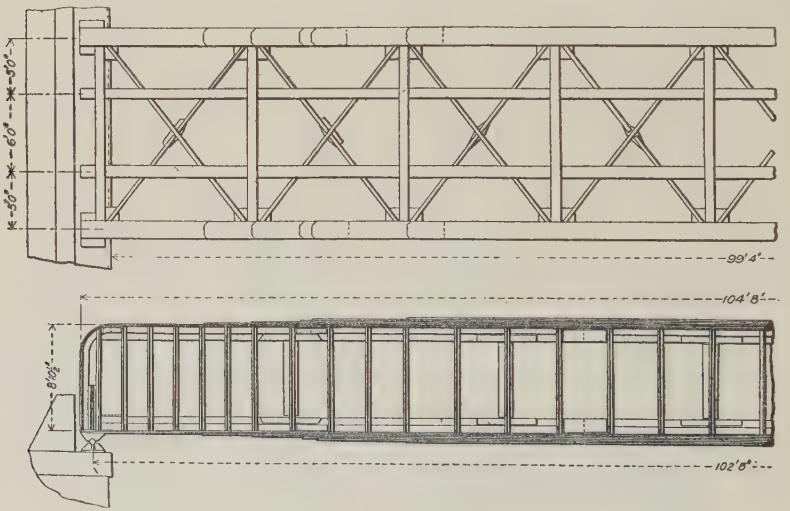


Fig. 93.

extension on the upper edge to receive the pin. The complete span weighs 160 tons.

Bridge No. 1,057 at Ashley Falls, Mass., on the Berkshire division, is an example of standard construction of a regular

long-span single-track through plate-girder bridge. It has a clear span of 99 ft. 4 in., and the girders are about 9 ft. deep over all, and 16 ft. apart on centers. The web has a uniform thickness of $\frac{7}{16}$ in., and is made in lengths of about 12 ft. 2 in. spliced for both shearing and bending moment stresses. The T-shape flanges are alike, top and bottom, and are made with 8×8-in. angles, side plates and cover plates with splices arranged as shown in the general plan and elevation.

The top flange angles do not extend quite to the ends of the girders, but are cut to clear the rounded upper corners of the girders. Their ends are cut square and abut against the ends of the end vertical web-stiffener angles of the same size, which, being much shorter, are very much more easily bent to fit the curved corners, and are spliced to the flange angles by the cover plate, and by a slightly curved short cover angle on each side of the web. The end panel of the web plate is reinforced by a $\frac{3}{8}$-in. plate riveted to each side and by four half height vertical angles over the center of the bearing on the shoe. The floorbeams vary from those already described, in that the web is made in three pieces, the two end sections projecting beyond the top flanges of the floorbeam to the tops of the girders, and making a kneebrace integral with the beam, thus affording a very rigid transverse bracing and abundant connection for the floorbeam reaction.

The end bearings of the girders are on cast-steel shoes 102 ft. 8 in. apart on centers, which, like their corresponding pedestals, each have one transverse and three longitudinal webs, all 2 in. thick. The transverse web is integral with a semicylindrical socket which gives continuous bearing to the hinge pin and reduces the stresses on it to simple compression. The edges of the bearing are beveled to give a minimum clearance of $\frac{1}{2}$ in. between shoe and pedestal, and the bearings are recessed $\frac{1}{2}$ in. at each end to engage the collars turned on the ends of the pin to lock it in place instead of the center collar used in some other cases.

The segmental rollers at the expansion end of the bridge have no guide ribs, but their ends are finished to fit accurately against guide ribs planed on the edges of the pedestal and bedplate surfaces. The bed plate is of cast steel 2 in. thick, with $5 \times 3\frac{1}{2}$-in. angles bolted on all sides to inclose the rollers and protect them from dust, etc. At the fixed end of the span the pedestals are seated directly on the masonry which is built

up about 8 in. higher than at the other end so as to allow for the bed plate and rollers. All rivets are $\frac{7}{8}$ in. diameter, with a minimum staggered pitch of 2 in. The span weighs about 145 tons.

Bridge No. 100 of the New York division forms part of the approach to the four-track Scherzer rolling lift bridge at Bridgeport, Conn., and has some special features differing from the usual construction for independent bridges. It has two spans, each with four girders 87 ft. $10\frac{1}{2}$ in. long over all. These girders are arranged in pairs, each pair forming an independent double-track span. The girders are 15 in. and 29 ft. 3 in. from the axis of the bridge, thus spacing the center lines only 30 in. apart, c.c. The web plates are 120 in. deep, and from $\frac{1}{2}$ in. thick and 14 ft. $4\frac{3}{4}$ in. long in the center panels to $\frac{5}{8}$ in. thick and 15 ft. 1 in. long in the end panels where they are reinforced by side plates to a maximum thickness of $3\frac{1}{4}$ in. and farther stiffened by half-length vertical angles interpolated between the regular web-stiffener angles. The web splices are made at the panel points where the floorbeams are connected, and each has 140 rivets in six vertical rows through two 21 × $\frac{1}{2}$-in. plates.

The T-shape flanges are unlike top and bottom, and at the top are made with two 8 × 8 × $\frac{7}{8}$-in. angles and seven 18-in. cover plates without side plates. The first cover plate only is full length, but at the center of the girder they aggregate $4\frac{7}{16}$ in. in thickness, all but one of them being $\frac{5}{8}$ in. thick. Each flange angle is made with one piece, 36 ft. 9 in. long, and one piece, 51 ft. $1\frac{1}{2}$ in. long, with joints staggered on opposite sides of the web, but opposite each other in the top and bottom flanges, and spliced with pairs of 8 × 8 × $\frac{3}{4}$-in. cover angles one panel long, sheared to $7\frac{1}{8}$ × $7\frac{1}{8}$ in. The bottom flange cover plates are 20 in. wide, the first one is full length, in three pieces, spliced by the projecting ends of the outer plates. Three of the seven cover plates are $\frac{5}{8}$ in. thick, and the remainder are $\frac{9}{16}$-in. thick.

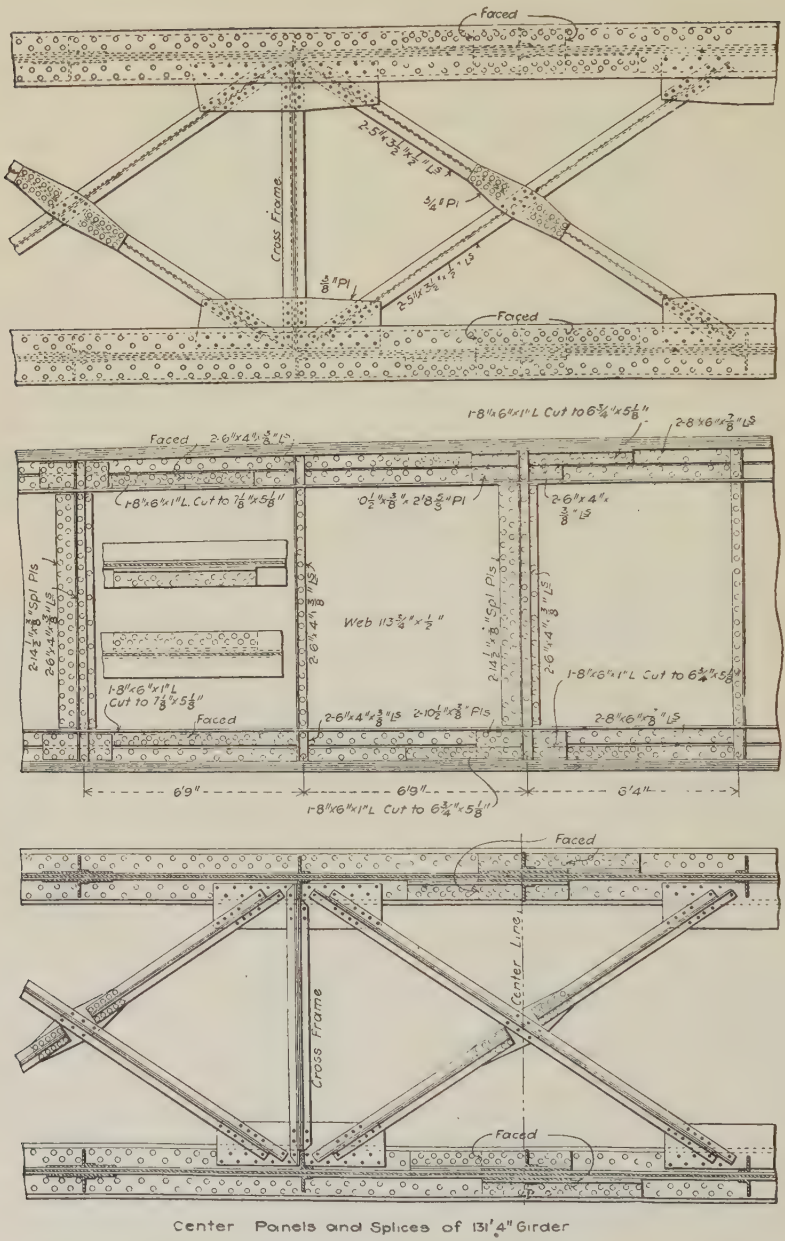
The girders have standard hinge bearings made with 6-in. pins engaging half holes in cast-steel shoes and pedestals 85 ft. $3\frac{3}{4}$ in. apart on centers. A 21 × $\frac{3}{4}$-in. planed bearing plate about $2\frac{1}{2}$ ft. long, called a "Wear" plate, is riveted to the bottom flange to receive the top of the shoe casting, and has four holes for $1\frac{1}{4}$-in. turned bolts. At the fixed ends the pedestals are seated on the masonry, but at the expansion ends, which are adjacent in

both spans, they rest on nests of nine 6-in. segmental rollers seated on 1½-in. rolled bed plates 6½ ft. long, which extend across the top of the pier and receive the rollers of both adjacent spans.

The floorbeams are 27 ft. 9⅝ in. long over all, and have webs made of single 54 × ⅝-in. plates reinforced at each end, with two 42 × ⅜-in. plates about 5½ ft. long. The top flanges are made with a pair of 6 × 6 × ¾-in. angles and one full length and two shorter 13 × ⅝-in. cover plates. The bottom flanges are made with two 6 × 6 × ¾-in. angles and one full length 15 × ⅝-in., one 15 × ⅝-in., and one 15 × 1 9/16-in. cover plates. The girder connection at each end is made with fifty 1-in. rivets through the end vertical angles and with a solid-web kneebrace over 5 ft. high, field-riveted to the top flange of the floorbeam. The stringers are made with a 32 × 1 7/16-in. web plate and have two 6 × 6 × ½-in. angles in each flange. Each stringer-floorbeam connection has twenty-two rivets, besides which the lower flanges of the stringers are seated on shelf angles with their horizontal flanges reinforced by pairs of vertical distribution angles shop-riveted to the floorbeam web. Horizontal connection plates are riveted to the webs of the girders and floorbeams in the planes of the lower flanges of the stringers and receive the single X-brace angles which form the lateral system. Each girder weighs about 48 tons, and one double-track span weighs about 675 tons.

The Erie Railroad.

Bridge No. 17 of the Mahoning Division of the Erie Railroad is a single-track deck span with two plate girders 131 ft. 4 in. long 9 ft. apart on centers, and about 10 ft. deep over all in the center, which carry a track with a curvature of 3 degrees, 30 minutes. The span is among the longest, if not the longest, of any railroad plate girder in this country, but is exceeded by the double-track Piallee bridge of 170 ft. clear span on the Eastern Bengal Railroad, which was described in "The Engineering Record" of June 2, 1894, and by the 213 ft. Mill Street highway bridge at Cincinnati, described Sept. 28, 1901. This bridge crosses Yankee Run, near Hubbard, Ohio, at a clear height of 12 ft. from base of rail to high water, and has a clear span of 125 ft. between abutments. The weight is approximately the same as that of a through truss bridge of equal length, but the plate girders were adopted because of the greater rigidity and capacity for



Center Panels and Splices of 131'4" Girder

Fig. 96.

with 44 ft. wheel base followed by a train load of 4,000 lb. per linear foot, with axle spacing as shown in the diagram, for a longitudinal force at the rails of 2 per cent of the maximum live load, for a transverse force of 600 lb. per linear foot from wind pressure, and for variations of 150° F. in temperature. The girders were designed according to the standard specifications of the Erie Railroad, and all steel more than $\frac{5}{8}$ in. thick was drilled from the solid or sub-punched and reamed.

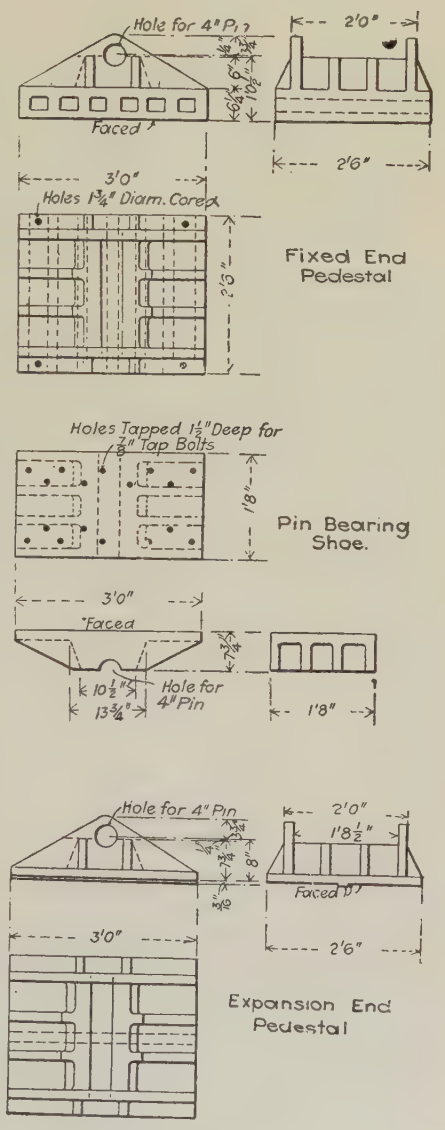


Fig. 97.

The girder web is made, in the center, of $\frac{1}{2}$-in. plates about 13½ ft. long, and at the ends of $\frac{9}{16}$-in. plates about 12 ft. and 19 ft. long. The top flange is made $\frac{1}{8}$ in. longer than the bottom flange in every 10 ft. to provide for the camber, which amounts to about 1 in. at the center for the unloaded girder. In the end panels the web plate of the girder was assembled so that it would at first project slightly beyond the backs of the bottom flange angles and insure full bearing over the shoe after being chipped. Elsewhere the web plates are ordered $\frac{1}{4}$ in. scant of the back-to-back depth of the girder so as to clear the flange cover plates.

All web splices are uniform, and are made with four vertical rows of rivets in two $14\frac{1}{2} \times \frac{3}{8}$-in. cover plates extending only to

the inner flange angles, and by four horizontal rows of rivets in a pair of $10\frac{1}{2} \times \frac{3}{8}$-in. plates between the inner and outer flange angles which splice the points for flange stresses. Pairs of vertical web-stiffener angles at every splice and intermediate between each pair of splices, divided the web into panels 6 ft. 9 in. long in the center and intermediate parts, and 5 ft. $3\frac{1}{4}$ in. long at the ends except over the bearings, where there are three pairs of stiffeners close together to provide for the maximum end shear.

The flanges have H-shape cross-sections and are duplicate top and bottom, each being made with four $6 \times 8 \times \frac{7}{8}$-in. angles in two pieces, and four $20 \times \frac{3}{8}$-in. flange cover plates. The outside angles and the first cover plate extend to the end of the girder; the inside angles are about 6 ft. shorter, terminating about even with the shoes. The shortest cover plate is $64\frac{1}{2}$ ft. long, its ends overlapping the ends of the pieces making up the long plate next to it, so as to form its splices. The splices in the flange angles are all staggered, as shown in the elevation, and are arranged to come as near as possible to the center of the girder, being symmetrical about the center point, except as they are right and left-handed. Each joint is made with a $6 \times 8 \times 1$-in. cover angle 4 ft. 10 in. long, with the back rounded to fit the fillet, and the flange cut to $6\frac{3}{4}$ and $5\frac{1}{8}$ in. The web-stiffener angles are faced to bearing at both ends on the inner flange angles, and short angles of the same size are fitted between the flange angles at every panel to continue them across the full depth of the girder.

Cast-steel shoes, 128 ft. 4 in. apart on centers, are secured to the bottom flanges with nine 1-in. tap bolts. The shoe has a massive transverse web about 6 in. wide acting like a pedestal with a full-length half-hole bearing for the 4-in. pin. The web has a $\frac{1}{2}$-in. clearance with the corresponding pedestal web, and is braced by three longitudinal webs, the outer two of which have full-hole bearings for the pin, and are received in clearance slots provided in the pedestal web. The pedestal corresponds in general to the shoe, inverted, and is higher so as to provide for a larger base to distribute the load and reduce unit masonry pressure to the required limits. The outside webs have full pin holes, thus locking shoe and pedestal together, and clear the shoe webs. They are braced on the outsides by pairs of transverse webs. In both shoe and pedestal all webs except the center transverse one are uniformly $1\frac{1}{4}$ in. thick. The shoes

weigh 900 lb. each; the fixed end pedestals, which are seated directly on the cut stone masonry, weigh 2,125 lb. each. The expansion and shoes weigh 1,215 each, and have their bottoms planed with a center longitudinal guide strip engaging the nest of rollers on which it is seated. The rollers rest on riveted beds made with double plate, the lower one projecting beyond the longitudinal edges of the upper one to receive four short $1\frac{1}{4}$-in. stone bolts with wedges. The bed plates are designed to distribute the pressure uniformly on the masonry without developing stresses of more than 12,000 lb. per square inch.

The girders are connected by the top and bottom lateral systems and by ten sway-brace frames made, as shown in the general cross-section, with pairs of angles back to back, riveted to large transverse vertical plates notched to clear the inner flange angles and field-riveted to the vertical web-stiffener angles at alternate panel points. Each panel between sway-brace frames has, in the planes of the top and bottom outside flange angles, two panels of X-brace lateral diagonals, each made with a pair of angles back to back, field-riveted to horizontal connection plates field-riveted to the flange angles. In all cases the angles in one diagonal of each panel are continuous, and those of the other diagonal are cut to clear them at the intersection and spliced there by a flange plate shop-riveted to the short angles and field-riveted to the long angles, the different diagonals having their flanges turned in opposite directions so that the other flanges will be in the plane of the connection plate.

The Baltimore & Ohio R. R.

The standard designs adopted by the Baltimore & Ohio Railroad for long-span double-track through plate-girder bridges is illustrated by a bridge over Patapsco River which is $85\frac{1}{2}$ ft. long on centers of bearings. There are two girders 88 ft. long and 9 ft. deep back to back of angles and 29 ft. apart on centers. They are skewed 16 ft. 9 in. and have one end panel of that length, and each has one skew panel and four regular panels of 17 ft. $2\frac{1}{4}$ in. as shown in the diagram which also indicates the lower lateral bracing and the stringer bracing.

The web plates are made in full panel sections $\frac{7}{16}$ in. thick at the ends and $\frac{3}{8}$ in. thick at the middle, spliced at floorbeam connections by pairs of $21 \times \frac{5}{8}$ and $\frac{3}{8}$-in. cover plates with six vertical

rows of rivets. The flanges are made with continuous $8 \times 8 \times \frac{7}{8}$-in. full length angles, $20 \times \frac{9}{16}$-in. cover plates and $12 \times \frac{5}{8}$-in. sideplates. The first cover plate is made in two pieces spliced by an outside cover plate $9\frac{1}{2}$ ft. long. All the other cover plates are unspliced, but the side plates are each made in three lengths partly spliced by a $4 \times \frac{7}{8}$-in. plate outside the flange angle. The top flange angles are butt jointed 4 ft. 1 in. from each end of the girder to $8 \times 8 \times \frac{9}{16}$-in. end angles which serve as vertical web stiffeners and are bent to fit the curved upper corner of the girder. They are spliced with a cover angle fillered out to make the difference with the thick flange angles.

The floorbeam connections are made with 52 field rivets in four vertical rows at each end. The lower flanges of the floorbeams are attached to the lower flanges of the girders by hori-

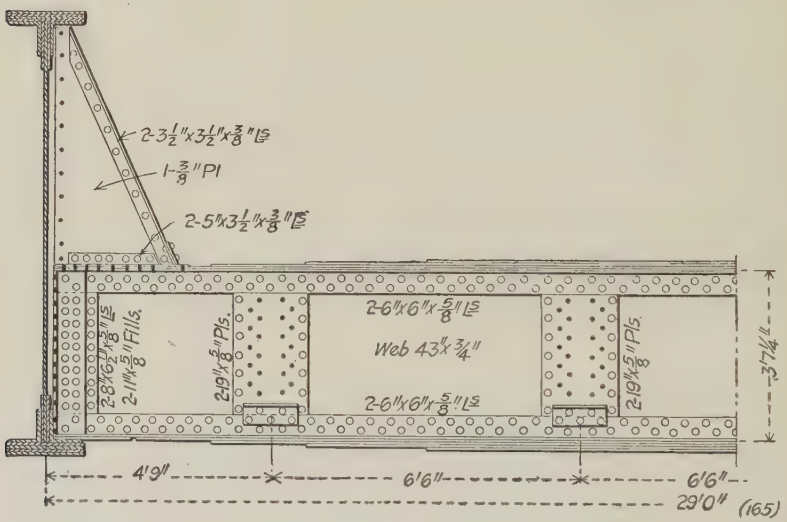


Fig. 99.

zontal lateral connection plates field-riveted to both members. A pair of vertical web-stiffener angles are shop-riveted to the girder web above the top flange of the floorbeam, and receive between them the field-riveted triangular gusset plate which is also field-riveted to the top flange of the floorbeam and serves for sway-bracing. The outsides of the girder webs are stiffened by single vertical angles opposite the pairs of floorbeam connection angles and by two pairs of single angles at points intermediate

between floorbeam connections. The floorbeams have pairs of shop-riveted web reinforcement plates at each stringer connection, and the stringer is seated without riveting on a horizontal shelf

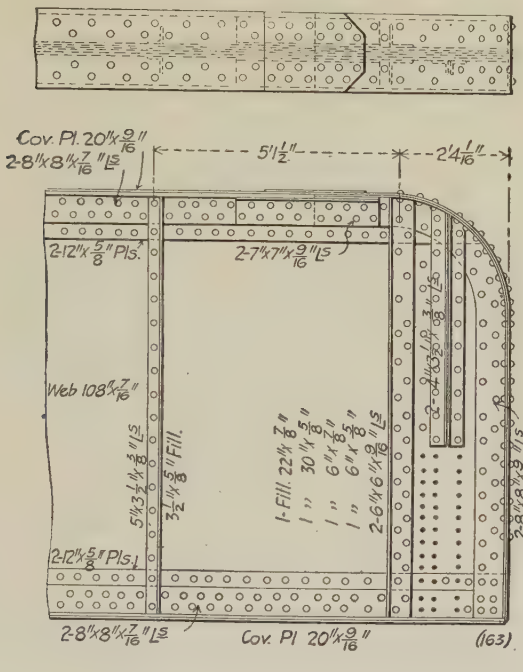


Fig. 100.

angle which promotes the convenience of erection and has 22 field rivets at each end.

All rivets are $\frac{7}{8}$ in. in diameter and have holes punched $1\frac{5}{16}$ in. except in material more than $\frac{5}{8}$ in. thick which is punched $\frac{3}{4}$ in. and reamed to $1\frac{5}{16}$ in., and rivets in floorbeam connections for which the holes are also punched $\frac{3}{4}$ in. and reamed through iron templates to $1\frac{5}{16}$ in.

A single-track through square-end plate-girder span 106 ft. long on pin centers has details like those above described. It has two girders 9 ft. deep and $15\frac{1}{2}$ ft. apart on centers with a clear span of 102 ft. 8 in. The floorbeams are 10 ft. 10 in. apart, and each panel between them is X-braced by the lateral diagonals which are uniformly made with single $6 \times 4 \times \frac{1}{2}$-in. angles with ten rivets in each connection. The 33-in. floorbeams carry two lines of $21\frac{1}{2}$-in. stringers each $3\frac{1}{2}$ ft. from the bridge axis. The bridge

is proportioned for an E-50 locomotive of the 1901 specifications and for a total dead load of 1,950 lb. per lin. ft. The girders have a maximum calculated moment and shear of 9,275,300 and 338,550 lb. respectively. The flange section of 65 sq. in. is made up with two spliced $8 \times 8 \times \frac{7}{8}$-in. angles, one spliced $18 \times \frac{13}{16}$-in. full length cover plate, two $18 \times \frac{13}{16}$-in. cover plates 52 and 70 ft. long and one sixth of the section of the web plate, which has a uniform thickness of $\frac{1}{2}$ in. The web rivets in the flanges are pitched 3 in. in the first and second panels, 4 in. in the third panel, and 5 in. in the fourth and fifth panels.

The fixed ends of the girders are supported on 4-in. pins engaging half holes in cast-steel shoes and pedestals. The shoes are bolted to sole plates on the girder flanges and are symmetrical with double pairs of vertical end web-stiffener angles. They are made of the same width as the flange cover plates, and have continuous semi-cylindrical bearings with a center transverse web and three symmetrical longitudinal webs. The side webs have outer bosses tapered to fit tightly with the recessed cast-steel washers under the small nuts on the shouldered pins. The pedestals correspond to the shoes, reversed, except that they have wider and longer bases to distribute the load on the masonry, and that they have no center longitudinal web, the side webs being made considerably heavier than those of the shoe. They are made high enough to compensate for the rollers under the expansion pedestals, and have a $\frac{1}{4}$-in. clearance with the shoes.

Wabash Railway.

Bridge No. 156A of the Toledo division of the Wabash Railway over One Leg Creek illustrates the essential features adopted by that road for standard single- and double-track long-span through plate-girder bridges. It has two spans erected as a single-track structure but with one line of girders made to be used later for the center girders in a three-girder structure which will be secured by adding another outside girder and a second floor system. There are four girders 85 ft. long, 6 ft. $8\frac{1}{4}$ in. deep back to back of angles, and 13 ft. apart on centers. They are all equally skewed about 13 ft., and the center pier is parallel with the abutments and has fixed bearings for all the girders seated there which are free to expand and contract at their abutment ends.

The girders are made to the Wabash specifications of mild

steel, with rivet holes sub-punched and reamed, and have a solid floor of transverse I-beams supporting a continuous flat slab of reinforced concrete laid on top of their flanges and covered by a

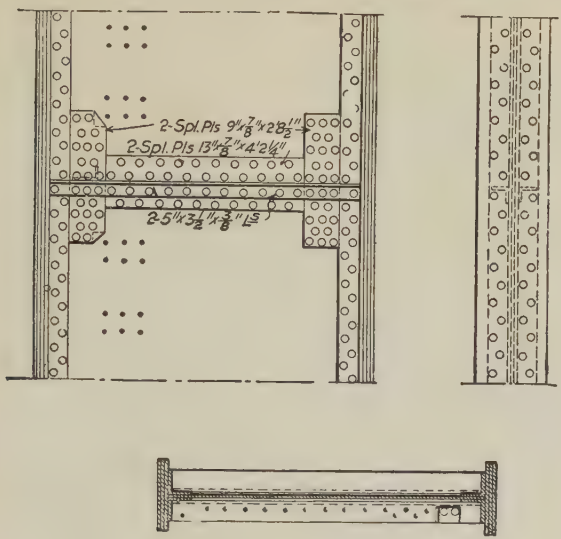


Fig. 101.

thick bed of stone ballast. The base of rail is 5 ft. $9\frac{1}{8}$ in. above the top of the masonry and 4 ft. $\frac{5}{8}$ in. above the backs of the bottom flange angles.

The girders are proportioned for a live load on each track consisting of a train weighing 5,000 lb. per lin. ft., preceded by

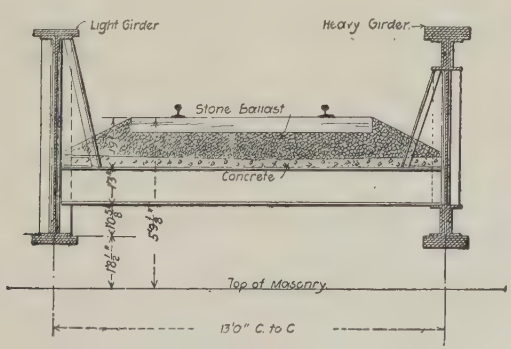


Fig. 102.

two coupled locomotives each weighing 355,000 lb. on a 48-ft. wheel base with 50,000 lb. on each of four axles 5 ft. apart.

The light and heavy girders are proportioned for a dead load of 2,850 and 4,700 lb. per lin. ft. respectively.

The girders overhang their shoes so that they are only 83 ft. long on centers of bearings, and the principal stresses and cross-sections computed are as follows: For the heavy girder, live load bending moment, 5,641,000 foot-pounds; dead load moment, 4,150,000; impact, 3,178,000; total, 12,969,000. Sectional area required at 16,000 lb. per sq. in. working stress, 124.74 sq. in. This was secured in each flange with four $6 \times 6 \times \frac{7}{8}$-in. angles, five $18 \times \frac{7}{8}$-in. cover plates, and two $10 \times \frac{7}{8}$-in. side plates with a combined gross section of 135.21 sq. in. Subtracting 19.21 in. for rivet holes and adding 9.6 in. for one sixth web area, the result is 125.60 sq. in. effective area. The maximum shears are 317,500, 200,000 and 194,800 lb. for live and dead load and impact, respectively, 712,300 lb. total, which is slightly more than 10,000 lb. per sq. in. for the $80 \times \frac{7}{8}$-in. web plate. The 42×42-in. shoe plate has a pressure of about 400 lb. per sq. in. on its bearing area of 1,764 in.

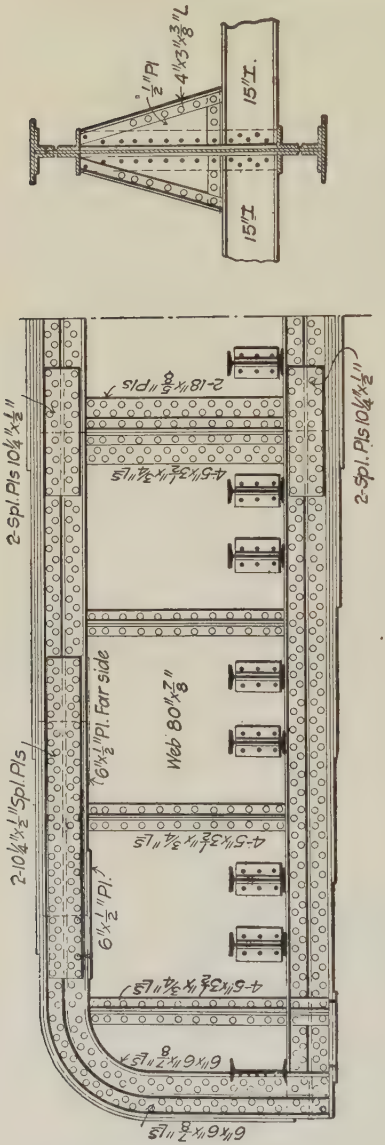


Fig. 103.

The 4-in. rollers have a pressure of 600 lb. per lin. in. The 15-in. 42-lb. floorbeams have moments of 403,200, 96,480, and 325,920 inch-pounds for live and dead loads and impact, and at 16,000 lb. per sq. in. working stress require 52 sq. in. sectional area against 58.9 in. actual.

The light girders have live, dead, and impact moments of 2,821,000, 2,075,000 and 1,626,000 lb. respectively, 6,522,000 total, and require 61.15 in. section. Each flange has two $6 \times 6 \times \frac{7}{8}$-in. angles and three $18 \times \frac{7}{8}$-in. cover plates with a combined cross-section of 66.73 sq. in. gross area; net area, 57.98, which, plus one sixth net web area, 4.82, gives an effective area of 62.80 sq. in. The maximum live, dead, and impact shears are 158,800, 100,000 and 97,000, total 355,800 lb. section required, 35.58 section of the $80 \times \frac{7}{16}$-in. web, 35.44 sq. in. Required area of shoe plate, 890 sq. in; actual area, 900 sq. in.

All girders have their upper corners square and protected by a bent cover plate at the pier end and curved to a radius of 30 in. at the abutment ends, where the first cover plate is cut 5 ft. $8\frac{1}{2}$ in. short and the milled ends are spliced with the overlapping end of the second plate.

The joints between the flange angles and the curved vertical end angles of the same size are made 6 ft. $11\frac{1}{2}$ in. and 9 ft. $2\frac{1}{2}$ in. from the end of the girder. The ends of the angles are milled, and the splices are made with two long vertical side plates and two pairs of short horizontal plates. The joints in the upper flange angles on one side of the web and the lower angles on the other side of the web are opposite and are spliced with the same plates. The lower flange angles are not spliced.

The web plates are about 16 ft. long at the ends of the girders and about 18 ft. long in the middle. The end and intermediate web plates are about 16 and 18 ft. long respectively. The end web joints are made with a pair of vertical $18 \times \frac{5}{8}$-in. cover plates with 70 rivets in six vertical rows, and two pairs of horizontal cover plates $10\frac{1}{4} \times \frac{1}{2}$-in. horizontal cover plates over the vertical legs of the flange angles. In the intermediate joints the horizontal plates are omitted and the $13 \times \frac{5}{8}$-in. vertical plates have only 48 rivets in four vertical rows. The web is divided into $4\frac{1}{2}$-ft. intermediate panels by double pairs of $5 \times 3\frac{1}{2} \times \frac{3}{4}$-in. angles, riveted back to back to make T-shape vertical stiffeners, one pair of which is on the center line of each web spliced.

The girders of each span are connected only by 15-in. 42-lb. transverse I-beams which support on their upper flanges the continuous reinforced concrete floor. At panel points these beams have the flanges on one side cut at both ends to allow the webs to engage the web stiffeners to which they are

field-riveted. Vertical plates with inclined outer edges are shop-riveted to the top flanges at both ends of these beams and field-riveted to the web stiffeners to make kneebraces reaching to the top flanges of the girders. In each panel two intermediate beams 18 in. apart are web-connected with six rivets to the girders. The short beams at the skew abutments have at one end of each, regular web connections to the long end girder, and at the other end are seated across the top flange of a 24-in. I-beam, parallel with the face of the abutment, which is web-connected to the girders with vertical bent plates.

In the light girders the top flanges have a T-shape cross-section and are in lengths of about 78½ and 74 ft., with their ends milled to join, at the second and third panel points, the horizontal portions of the end web-stiffener angles which are respectively about 12 and 16½ ft. long, and are more easily bent to fit the curved upper corner of the girder web than the long flange angles would be. The vertical flanges of these angles are not spliced, but the joint between them is covered by the web-stiffener angles. The first top flange cover plate has a length of 67 ft. 4¼ in., and its milled ends abut against those of the 13-ft. 7 3/16-in. cover plates which are bent 90 degrees to the same curve as the upper edge of the web plate and continue down over the end web-stiffener angles to the bottom flange. This joint is covered by the overlapping end of the second cover plate 76 ft. 3½ in. long which really splices the horizontal legs of the top flange angles.

The web is 7/16 in. thick throughout, and is made with one middle section about 27 ft. long and two 29-ft. end sections. The two joints are spliced with two 13 × ⅞-in. vertical plates having 42 rivets in four vertical rows, and two pairs of 9 × ⅞-in. horizontal plates, each pair having 24 or 26 rivets in four staggered rows. The floorbeam connections are the same as for the heavy girders, except that there are horizontal angle clips riveted to the web-stiffener angles to receive the bottom flanges of the beams.

All girders have cylindrical cast-steel bearings, about 81 ft. apart on centers, bolted to the bottom flange shoe plates. These engage cast-steel pedestals which at the fixed ends are anchor-bolted to the masonry, and at the expansion ends are seated on nests of rollers and bed plates. All girders were completely shop-riveted and shipped whole, ready for erection.

Chicago Great Western Railway.

Standard features of long-span deck plate-girder bridges on the Chicago Great Western Railway are illustrated by the 90-ft. Wapsie River bridge. The design differs little except in dimensions and materials from standard short spans. The girders are 90 in. deep, 92 ft. 2 in. long over all, and 8 ft. apart on centers. They are connected by six vertical transverse sway-brace frames, field-riveted to the vertical web stiffeners and to the flanges, and the panels between the frames are braced by pairs of zigzag lateral angles in the planes of the top and bottom flanges. The connections to the flanges are made with horizontal plates shop-riveted to the sway-brace frames and field-riveted by a single line of rivets to another plate, shop-riveted to the girder flange angle which is slotted when necessary to engage the web-stiffener angle.

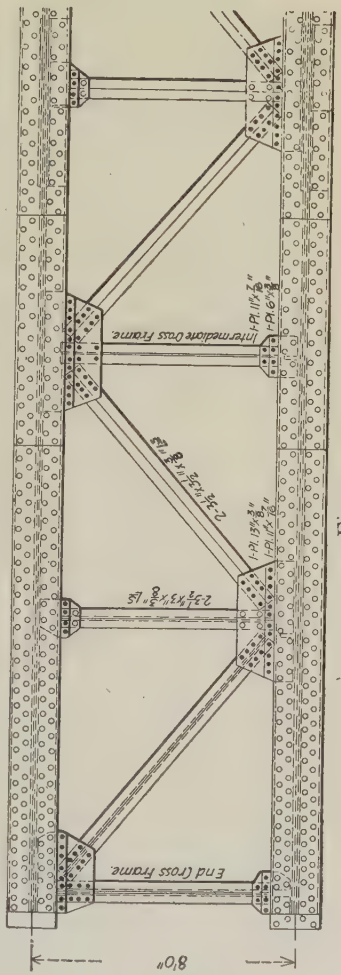


Fig. 104.

The girder webs are stiffened with intermediate vertical pairs of angles from about 3½ to 6½ ft. apart, all of which are crimped, except those receiving sway-brace frames which are fillered flush with the flange angles. At the ends the webs are reinforced to a total thickness of 2 in. and are stiffened by double pairs of larger vertical angles on the center line of the bearings. These pairs of angles are spaced ¾ in. apart in the clear so as to receive the sway-brace connection plates between their outstanding flanges. The web is ½ in. thick throughout, and is made in lengths of about 18 ft. with the joints uniformly spliced with a pair of 13¾-in.

cover plates with two rows of vertical rivets. The flanges have a maximum section of two $8 \times 8 \times \frac{3}{4}$-in. angles and three

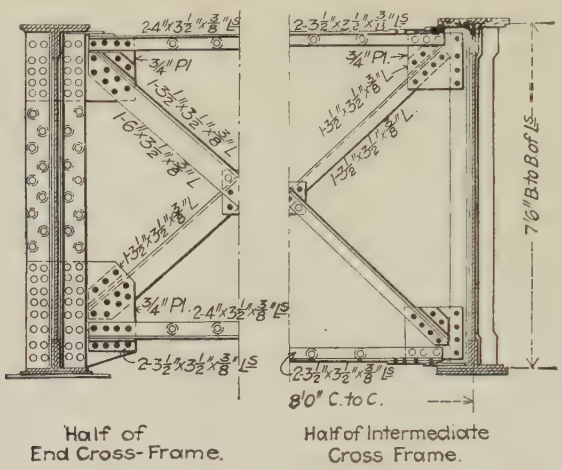


Fig. 105.

$18 \times \frac{5}{8}$-in. cover plates, of which the first one is the full length of the girder.

Each flange angle is made in two pieces, the joints being opposite each other in top and bottom flanges and staggered on opposite sides of the web. They are spliced with a single $8 \times 8 \times \frac{1}{2}$-in. angle planed down to $7\frac{1}{4} \times 7\frac{1}{4}$ in. on the joint side of the web, and by a single $7\frac{1}{4} \times \frac{1}{2}$-in. plate on the opposite side of the web, both being as long as will clear the adjacent web-stiffener angles. The top flanges project $5\frac{1}{2}$ in. beyond the ends of the web to form short cantilever brackets supporting the track ties over the masonry.

The $\frac{7}{8}$-in. end lower lateral plates are riveted across the bottom flange cover plates to form sole plates, with bearings on planed narrow raised transverse ribs on cast-steel pedestals with longitudinal and transverse webs and top and bottom plates. The girder is secured to each pedestal with two $1\frac{3}{4}$-in. bolts. The fixed end pedestal is seated on a cast-steel bolster $6\frac{1}{4}$ in. high with two transverse and five longitudinal webs and two 2-in. anchor bolts passing through the pedestal base. The expansion pedestal has two narrow transverse guide ribs on the under side of its planed base plate, which secure the nest of seven $3\frac{3}{4}$-in. rollers with bearings in cast-steel side bars to which are bolted

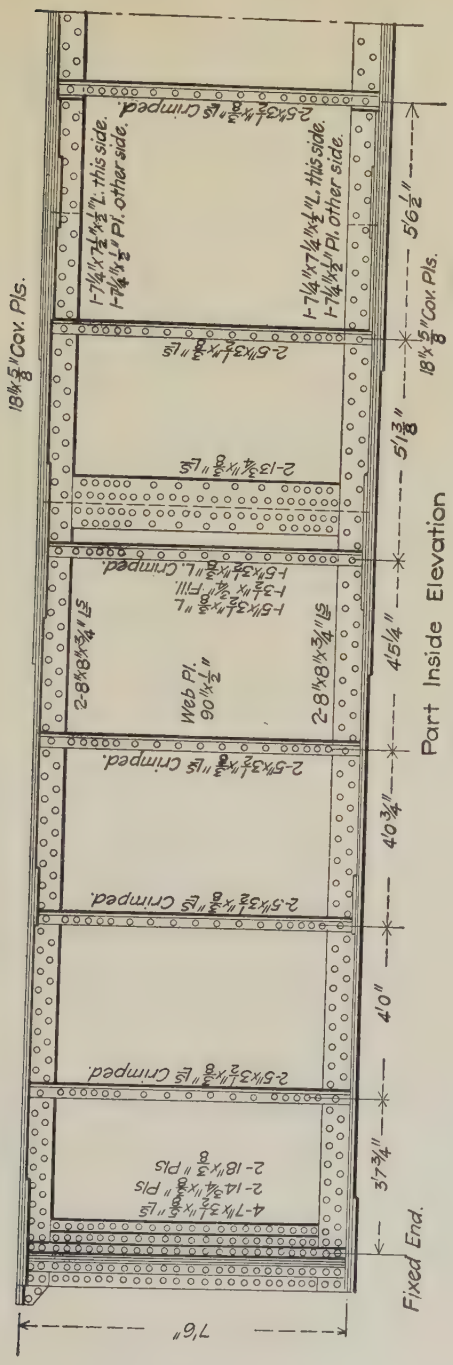

4-7½x3½"x⅝" L's
2-14¾"x⅜" Pls
2-18"x⅜" Pls
2-5½x3½"x⅜" L's Crimped.
2-5½x3½"x⅜" L's Crimped.
2-5½x3½"x⅜" L's Crimped.
2-8"x8"x¾" L's
Web Pl. 90"x½"
2-8"x8"x¾" L's
1-5"x3½"x⅜" L
1-3½"x3½"x⅜" Fill
1-5"x3½"x⅜" L. Crimped.
2-13¾"x⅜" L's
2-5½x3½"x⅜" L's
1-7¼"x7¼"x½" L. this side.
1-7¼"x½" Pl. other side.
2-5½x3½"x⅜" L's Crimped.
1-7¼"x7¼"x½" L. this side.
1-7¼"x½" Pl. other side.
18"-5⅛" Conv. Pls.
18"-5⅛" Conv. Pls.
7'6"
3'7¾"
4'0"
4'0¾"
4'5¼"
5'1⅜"
5'6½"
Fixed End.
Part Inside Elevation

Fig. 106.

horizontal transverse angles, forming with them a rectangular frame inclosing the rollers so as to protect them from dirt, etc. The rollers are seated on a cast-steel bed plate with guide ribs on the upper surface.

Chicago Junction Railroad.

Standard through-span, three-track, plate-girder structures have been designed and constructed for the Fortieth Street track elevation of the Chicago Junction Railroad. A typical structure is at the Armour Avenue crossing, where there are four 69-ft. girders 87 in. deep and 13 ft. apart on centers (Fig. 104). They are supported on masonry abutments 66 ft. apart in the clear, and are proportioned for a live load of 5,000 lb. per lin. ft. of track plus a concentrated load of 50,000 lb. Unit stresses are in accordance with Cooper's specifications, and are assumed to be 1.7 times as great for the inside as for the outside girders.

The end vertical angles are curved to correspond with the rounded upper corners, and are cut to butt against the square ends of the top flange angles. The ends of the webs are reinforced to a total thickness of $2\frac{3}{4}$ in., and have pairs of vertical angles. There are no other web-stiffener angles except at the transverse gusset plates, which sway-brace them to the floorbeams, 9 ft. $1\frac{1}{2}$ in. apart. The webs are made in three pieces, shop-spliced with four rows of shop rivets. The flanges are made with pairs of 6 × 6-in. angles with double rows of rivets in each leg, and are reinforced with 14-in. cover plates and with web plates. The latter are 12 in. wide, and have four rows of rivets for the top flanges, and are $22\frac{1}{2}$ in. wide and have seven rows of rivets for the bottom flanges. The first cover plates on the bottom flanges are 20 in. wide, and project beyond the flange angles to receive the floorbeam rivets. The flange angles and plates are made in single lengths, and the webs are spliced at the upper and lower edges with long plates. The floorbeams are spaced $18\frac{1}{4}$ in. apart, and are connected to the webs by long horizontal angles. There are no lateral angles or stringers, and the lower flanges of the girders have steel base plates planed to the grade of the track and seated on the planed tops of cast pedestals having outside longitudinal guide strips and stone-bolted to the masonry.

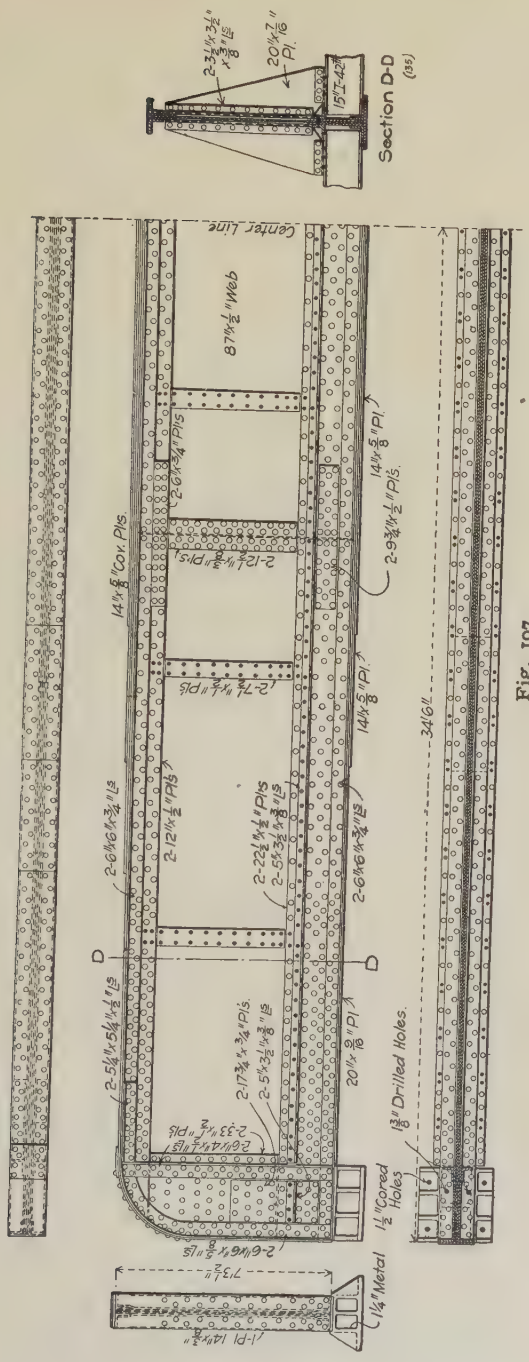

Section D-D
Center Line
87"x½" Web
14"x⅝" Cov. Pls.
2-6"x6"x⅝" L's
2-12"x½" Pls
2-6"x6"x⅝" L's
2-5½"x5½"x⅝" L's
2-6"x4½"x⅝" L's
2-33"x½" Pls
2-17¾"x¾" Pl's.
2-5"x3½"x⅝" L's
20"x⅜" Pl
2-6"x6"x⅝" L's
2-22½"x¾" Pls
2-5"x3½"x⅝" L's
2-7½"x¾" Pls
2-12½"x¾" Pls
14"x⅝" Pl
2-9¾"x½" Pl's.
14"x⅝" Pl
34'6"
1⅛" Drilled Holes.
1½" Cored Holes
1/4" Metal
1-Pl 12"x⅜"
7'3½"
2-6"x6"x⅝" L's

Fig. 107.

PART III

SHALLOW, SOLID, BALLASTED, AND CONCRETE FLOORS

PART III

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SHALLOW, SOLID, BALLASTED, AND CONCRETE FLOORS

VERY shallow floors have been adopted to a large extent within the last few years in the elimination of grade crossings where it is necessary to preserve the minimum clearance underneath, and a practically or absolutely continuous floor requires less space than a deeper and lighter one. Solid floors have also been strongly commended on account of the increased mass and inertia of the structure and their great rigidity. They permit greater freedom in locating and changing the position of the track rails, are safer in case of derailment, permit continuous ballasting, prevent changes in the roadbed, and afford complete protection from drip underneath the bridge. They are likely to be objectionable on account of their greater cost, the obstruction to light and air below the bridge, and the danger of corrosion unless special care is taken to protect them from moisture. At first they were built wholly of steel or with steel beams and girders decked with waterproofed planks. Rectangular troughs and I-beams with flat cover plates were the most frequent types, but various special shapes were adopted occasionally and sections were built up of channels, Z-bars, and other shapes.

Recently solid floors of concrete, or of concrete and steel, have been very carefully designed and adopted on some important railroads, and it is on these lines that development promises to be most rapid in the near future. There are two distinct classes of concrete floors, those in which reinforced concrete slabs bridge over considerable spaces between steel beams and girders, and themselves develop beam action, and those in which the beams and girders are spaced so close that there is very small bending moment on the mass of concrete between them which serves chiefly to provide a continuous surface and to completely inclose and thoroughly protect the steel work.

CHAPTER IX.

TROUGH, BEAM, AND PLATE FLOORS.

Recent Examples of Solid Floor and Shallow Floor Construction for Short Spans.

The 100-ft. overhead plate-girder span of the Wabash Avenue Bridge of the Chicago Junction Railway has not clearance for a ballasted floor, and the rails are laid directly on a continuous flat plate riveted to the top flanges of the floorbeams, spaced close together, and illustrating one of the types of shallow floor construction standard on this railroad. The first cover plate of the bottom flange of the main girder projects 3 in. beyond the flange angle and outer cover plates, and has two field-driven rivets in the bottom flange of each of the 12-in. 45-lb. floorbeams 15 in. apart on centers. The top flanges of the floorbeams are field-riveted to a continuous $5 \times 3\frac{1}{2} \times \frac{3}{8}$-in. horizontal angle field-riveted to the girder web, and at panel points $7\frac{1}{2}$ ft. apart they are riveted to gusset plates extending to the top flanges of the girders. The ends of the floorbeams are seated on the bottom flange angles of the girders and are not riveted to them, but are riveted through fillers to the outer edges of the wide flange cover plates. This floor weighs 53 lb. per sq. ft.

Where the bridges have one line of columns in the center of the street, and one line on each curb, the transverse girders support on their top flanges, longitudinal I-beams $20\frac{1}{2}$ in. apart on centers. This floor weighs $42\frac{1}{2}$ lb. per sq. ft.

In a paper on thin floors for bridges, presented to the American Society of Civil Engineers, in 1892, Mr. A. F. Robinson said: "Our problem is to design the thinnest possible bridge floor which will comply with the following requirements: first, it must be reasonably low in first cost, and must be long lived; second, it should not be liable to fail under derailment, and if possible it should be able to stand under collisions; third, the erection must not be unreasonably inconvenient, and the floor should be so arranged that it can be put in without interrupting the traffic; fourth, it should show low cost for maintenance and be so arranged that ties can be renewed with-

out serious expense or interruption of traffic; fifth, it must not attract unfavorable criticism of passengers by the way it carries a train; sixth, it should please train men and gain their confidence by its appearance of strength and ability to stand hard, continuous service."

In 1889 Mr. Robinson built a rectangular trough floor 12 in. deep, made of plates and angles. The troughs were riveted to the girder webs close to the bottom flanges, and had brackets on which the ties were supported. They were about 15 ft. long and $11\frac{1}{4}$ in. wide, made with $3 \times \frac{3}{8}$-in. angles and $\frac{3}{8}$-in. plates. A little later this floor was modified by reducing the depths of the troughs to 10 in. and spacing them 2 ft. apart on centers. This floor required twelve field-driven rivets per lin. ft., all of which were easily driven.

A corrugated patent floor used for the Gaspee Street Bridge, Providence, in 1895, and in other structures, has the troughs formed with special channel-like bottom sections, having a horizontal thick web, wide and thinner flaring flanges. The sides of the trough were made with special oblique angles with very wide inclined flanges riveted to the flanges of the bottom pieces, and narrow horizontal flanges riveted to $5\frac{5}{8}$-in. top plates connecting the troughs. The troughs weighed 37.4 lb. per sq. ft., and were shop-riveted in 8 × 12-ft. sections, and a wooden tie was set on stone ballast in each trough. The ends of the troughs were seated on continuous shelf angles shop-riveted to the girder webs, and reinforced by vertical angles under their horizontal flanges at bearings. The floor was claimed to be absolutely watertight and to have a minimum loss of strength from punching.

The New York Central & Hudson River Railroad has two standard types of solid floors, one made with rectangular transverse troughs, built with horizontal and vertical plates and angles, and web-connected to the main girders with wide flange vertical angles; and the other consisting of transverse I-beams web-connected to the main girders just clear of the horizontal legs of the lower flange angles and covered with continuous $\frac{7}{16}$-in. plates.

The Gokteik viaduct, Burmah, which was built by Americans, has a floor of $\frac{5}{16}$-in. flat plates, but the train loads are carried on wooden sleepers laid on the plates directly above the stringers.

Trough floors are usually filled with stone ballast reaching above the tops of the troughs, to receive the wooden ties clear

of the troughs, and the latter are drained or they are filled with cement or asphalt concrete, with the upper surface waterproofed and pitched to drains. Sometimes, however, ballast is omitted, as in the Frazer River Bridge, where the ties are laid on wooden fillers in the bottoms of the troughs.

One of the viaducts for the track elevation of the Delaware, Lackawanna & Western Railway in Newark, N.J., passes over the roof of an express building, and the structure was built to

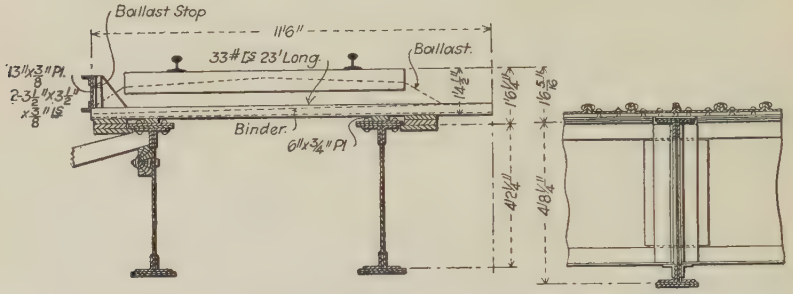


Fig. 108.

make the 35-ft. spans form part of the roof. This necessitated a solid waterproof floor, which was built with 12-in. transverse channels with their vertical flanges shop-riveted together and their horizontal webs field-seated on 6 × ¾-in. filler plates on the center lines of the top flanges of the girders to raise the channels clear of the flange rivet heads. The channels were shop-riveted together in sections as large as could be conveniently handled; and as their ends were secured to the girders by wooden filler and bearing pieces bolted to the former, the only field-riveting required for this floor was in the flange connections of the channel sections. The ends of the channels are stiffened by fascia girders riveted to gusset plates between the channel flanges. The upper sides of the channels are covered with asphalt binder pitched from the center to the ends. The floor is covered with stone ballast in which the ties are laid in the usual manner.

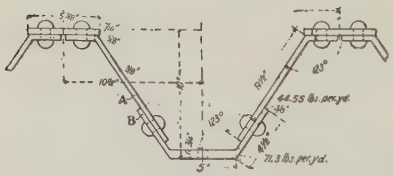


Fig. 109.

Trough construction for the floors of the Chicago Track

Elevation spans is illustrated by a 631-ft. viaduct 155 ft. wide with skew plate-girder spans carrying nine tracks across the intersection of the Southwestern Avenue Boulevard and West Thirty-ninth Street.

The spans have no floorbeams, stringers, or lateral systems, and the girders are connected only by the transverse plate and angle troughs of the solid-ballasted floor system. The troughs are $16\frac{1}{2}$ in. wide on centers of webs and $13\frac{1}{2}$ in. deep, made with $\frac{1}{2}$-in. plates and $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$-in. angles, and are shop-riveted together in sets of three, four, or five, so as to form sections up to about 9 ft. long. These sections are connected by bottom plates field-riveted to one section and shop-riveted to the adja-

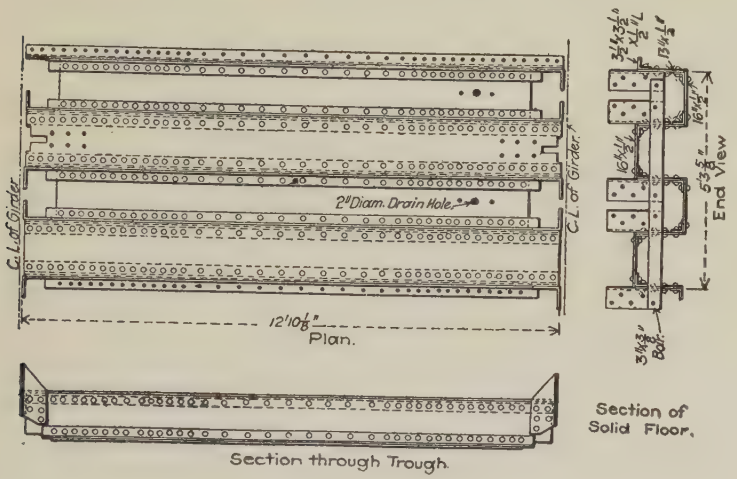


Fig. 110.

cent section. The troughs are made to fit closely against the main girders, the ends of their lower angles being cut to clear the edges of the flange angles, and to take bearing on the upper surfaces of the flange cover-plates. The troughs are connected to the girders by short vertical angles shop-riveted to the end of each web plate with five rivets, and field-riveted to the girder web with five rivets. All rivets are $\frac{7}{8}$ in. diameter, and the holes for field-driven rivets were punched $\frac{3}{4}$ in. and reamed in the field after assembling. The under side of the troughs only was painted in the shop with one coat of boiled linseed oil. After erection the troughs were filled with asphalt concrete rounded up to a height of about $2\frac{1}{2}$ ft. on the girder webs, where it

makes a smooth-curved fillet reaching to the top flanges of the troughs and sloping thence about 4 in. to short $1\frac{1}{2}$-in. vertical pipes, projecting through the bottom plates of the troughs at one end with their tops flared out to receive drainage at the surface of the concrete. They discharge into copper gutters attached to the under sides of the troughs and connected with leaders on the columns. The concrete filing has an asphalt surface $1\frac{1}{2}$ in. thick, and is covered with broken stone ballast, in which the 7×7-in. ties are laid with their upper surfaces $13\frac{1}{2}$ in. above the bottom plates of the troughs.

The trough floor is designed for a dead load of 3,500 lb. and a live load of 5,000 lb. per lin. ft. of track, plus a concentrated live load of 50,000 lb., the uniform live load being assumed to be distributed over a width of 8 ft. transversely, and the concentrated load over 8 ft. transversely and 4 ft. longitudinally. The maximum total moment thus produced is 711,138 in.-lb., and the corresponding cross-sectional area is 21 sq. in. The end shear in the trough is 13,800 lb., and the flange rivets are pitched 3 in. for 4 ft. at each end and 6 in. in the middle. All material is of medium open-hearth steel except the soft open-hearth steel shop rivets and the wrought-iron field rivets. All rivet holes are sub-punched and reamed.

Several types of solid floor are in service on the Chicago & Northwestern Railway. In the Mayfair cutoff for the Chicago track elevation built in 1902, short three-track deck spans over Grand Avenue and North Avenue for both square and skew crossings have longitudinal 20-in. I-beams from 16 in. to 34 in. apart seated on the top flanges of the main girders, and decked with a flat plate $\frac{3}{8}$ in. thick. Fascia girders 20 in. deep are knee-braced on each side above the deck plate to retain the broken stone ballast $15\frac{1}{4}$ in. deep to the upper surfaces of the ties. The deck is pitched $1\frac{1}{2}$ in. from the center of the street to the abutment for drainage. The floor is proportioned for locomotives with four 40,000-lb. axle loads.

All metal work exposed to the ballast was thoroughly cleaned, dried, and in cold weather heated by application of hot sand and painted with hot asphalt free from coal tar and coal-tar products. The specifications required that asphalt must not flow at a temperature of less than 325° F.; must not be brittle at − 15° F. when spread thin on glass; must not lose more than one per cent by volatilization in two hours at 275° F.; must resist the action of salt water of a 25 per cent solution of sulphuric acid and of

a 10 per cent solution of ammonia. After painting, the surface was covered with 1 in. of 1 : 4 hot asphalt and torpedo sand spread with hot tamping irons and covered with a $\frac{1}{4}$-in. coat of asphalt heated to 350 degrees, smoothed with hot irons and sprinkled with fine sand.

The Armitage Avenue, West Fullerton Avenue, and West Wrightwood Avenue through span-crossings of the Mayfair cut-off have similar floors, except that the deck plates are carried by 15-in. 60-lb. I-beams 13 ft. long and $17\frac{5}{8}$ in. apart, with their ends web-connected to the main girders and seated on their bottom flange angles. Longitudinal angles are field-riveted to the girder webs and to the upper surfaces of the deck plates.

The double-track through bridge over the Vine Avenue subway has 15-in. 80-lb. transverse I-beams 3 ft. $2\frac{7}{8}$ in. apart on

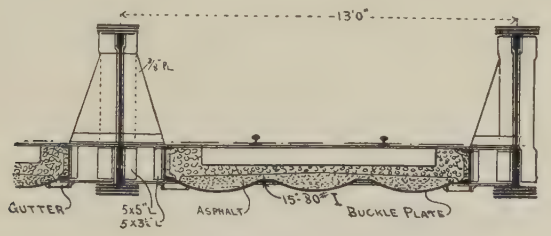


Fig. 111.

centers, web-connected to the main girders as shown in the part cross-section. The floorbeams are connected near both ends by continuous lines of 15-in. longitudinal channels coped to tight fits flush top and bottom, and serving to retain the ballast carried on the solid floor of $\frac{5}{16}$-in. buckle plates in transverse three-panel sheets field-riveted to the bottom flanges of the floorbeams. All surfaces exposed to the ballast are protected by a $\frac{1}{2}$-in. coat of pure mineral rubber leveled up with a filling of concrete made with clean hot sand, gravel, and sufficient mineral rubber to fill all voids.

The concrete is crowned from the axis of the bridge and pitched to drain holes at each end of each panel, which are protected by a dozen or more large pebbles to maintain a free opening and prevent clogging by the ballast of broken stone or screened gravel. The upper surface of the concrete is finished with pure mineral rubber 1 in. thick at the drain holes and $\frac{1}{2}$ in. thick elsewhere, smoothed with hot irons and sprinkled with fine sand. The drain holes discharge into gutters made with

8-in. horizontal longitudinal channels, webs down, riveted to the lower flanges of the I-beams on both sides of the bridge. Each floorbeam is proportioned for a concentrated load of 22,500 lb. on each rail and for 7,250 lb. total dead load. This construction gives a very shallow ballasted floor, and has been extensively used.

On some of the 40th St. Chicago track elevation the ballasted floor has been made with transverse I-beams covered with $2\frac{3}{8}$-in. longitudinal splined boards essentially corresponding to the shallow floors in use on the Chicago & Western Indiana Railway, and the joint track elevation since 1901. For the joint track elevation work the 12-in. I-beams weigh from 40 to 55 lb., and are spaced from 13 to 16 in. on centers. These give a metal weight of 600 lb. per linear foot of single track. For the Chicago & Western Indiana work the 15-in. 45-lb. I-beams are spaced 18 in. apart and, together with their braces, weigh 440 lb. per linear foot of single track. In both cases the edges of the wooden floor are sloped upwards and supported on wedge-shaped pieces that carry them above the longitudinal angles riveted to the girder webs. Several other types of solid floor

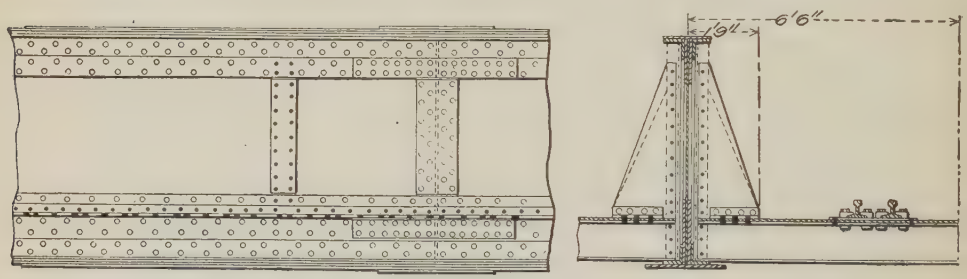


Fig. 112.

have been used for Chicago track elevation during the last few years. That adopted by the Pittsburg, Fort Wayne & Chicago Railroad in 1897 has transverse rectangular troughs made entirely of plates and angles forming a zigzag line in cross-section. It weighs 830 lb. per linear foot of track, and is field-riveted with web-connection angles to wide vertical plates in the flanges of the main girders. The girders are braced by gusset plates stiffened with inclined angles and connected by bent plates to the summit of every fourth trough.

The type used by the Chicago, Milwaukee & St. Paul Railway in 1899 has 12-in. 45-lb. transverse I-beams 15 in. apart, which

rest on the lower flanges of the main girders and are covered by continuous flat deck plates $\frac{5}{16}$ in. thick, over which there is laid under each rail a line of 10-in. channels and an $8\frac{1}{2} \times \frac{1}{4}$-in. bearing plate. This floor weighs 820 lb. per linear foot of single track.

The shallow floor used by the Chicago, Burlington & Quincy Railroad in 1900, has 12-in. 45-lb. transverse I-beams $14\frac{1}{2}$ in. apart on centers, which are seated on the angles of the regular T-shaped bottom flanges of the main girders, and are connected to the latter by pairs of vertical angles shop-riveted to the girder

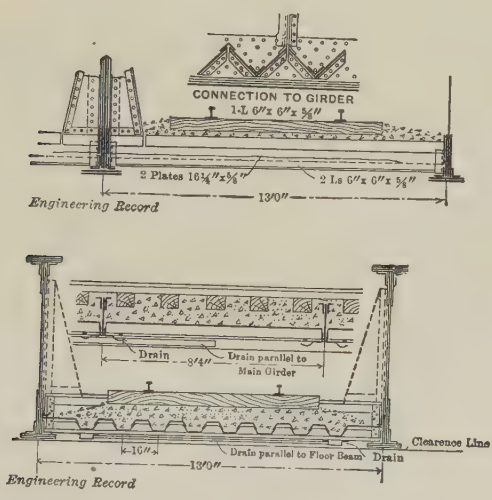


Fig. 113.

webs and bolted to the floorbeams. The beams are covered with $\frac{5}{16}$-in. deck plates, above which there is a trough for each rail, consisting of a $20 \times \frac{1}{2}$-in. plate, two $5\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$-in. angles, and $2 \times 3 \times \frac{5}{16}$-in. outside angles. The rails are laid in these troughs on insulation $\frac{1}{8}$ in. thick. Every sixth beam has gusset-plate connections to the main girders. The floor weighs 870 lb. per linear foot of track.

The Chicago, Rock Island & Pacific Railway standard of 1900 also has a plate-and-beam floor, but differs materially from the others in the connections of the beams to the main girders. The floor plates are $\frac{3}{8}$ in. thick, and the 12-in. 45-lb. beams are $15\frac{1}{16}$ in. apart; their ends are beveled and have inclined connection angles riveted to them and to bent plates field-riveted to the

girder webs through the vertical flange plates and through horizontal angles on top of the deck plates. The first cover plate in the girder flange is special, projecting beyond the edges of the outside cover plates and bent downwards to engage the ends of the floorbeams. The rails are laid on $16\frac{1}{4} \times \frac{3}{8}$-in. rail plates with $6 \times 4\frac{1}{2}$-in. angles on the outsides only. Every

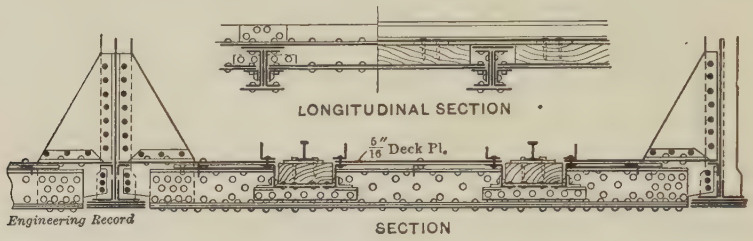


Fig. 114.

fourth floorbeam is braced to the girders by gusset plates with their lower corners cut to clear the longitudinal angles on the webs. This floor construction weighs 890 lb. per linear foot of track, and was designed for a 50,000-lb. axle load plus 25 per cent for impact, each floorbeam being proportioned to carry

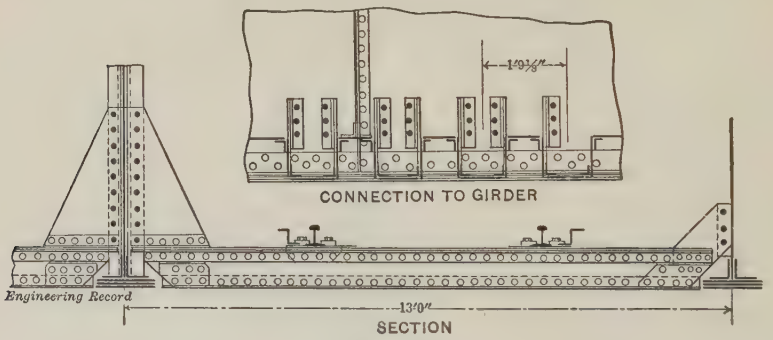


Fig. 115.

one-third of this amount with maximum unit stresses of 10,900 lb.

The Chicago, Rock Island & Pacific type of 1897 has rectangular transverse floor troughs made of $9\frac{1}{2} \times \frac{3}{8}$-in. vertical plates, $9\frac{7}{8} \times \frac{7}{16}$-in. top plates, $11\frac{7}{8} \times \frac{7}{16}$-in. bottom plates, and

3 × 3 × $\frac{3}{8}$-in. angles. The ends are beveled to clear the flange cover plates of the girders, and they are connected to the girder webs by oblique 10 × $\frac{7}{16}$-in. plates field-riveted to the trough webs and to the girder webs. The girders are kneebraced to every third trough by deep gusset plates, connected to the top plate of the trough independently of the regular trough connections to the girder. The rails are seated on continuous 18 × $\frac{1}{2}$-in. track plates, each provided with a single 5 × $3\frac{1}{2}$ × $\frac{1}{2}$-in. outside Z-bar acting as a guard rail. The weight of the floor is 875 lb. per foot of single track.

In 1899 a shallow floor made with 15-in. transverse I-beams $15\frac{7}{8}$ in. apart on centers was substituted for the above type by this road for the track elevation. The connections of the I-beams to the girders were of a special character, designed to bring the lower flange of the I-beam flush with the lowest point of the girder flange and thus reduce to the utmost the depth of the floor. The lower corners of the I-beams were beveled to

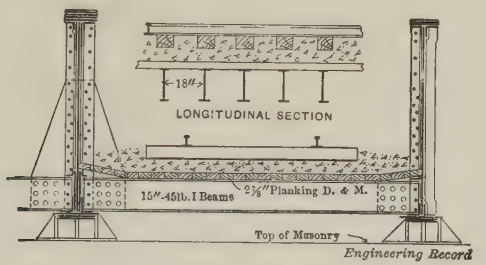


Fig. 116.

clear the girder flanges, and to their webs were shop-riveted inclined connection angles with bent plates field-riveted at the lower ends to corresponding continuous bent plates on the girder flanges, and at their upper ends field-riveted directly through the girder webs. The girders were kneebraced to every fifth beam with gusset plates. The top flanges of the I-beams were covered with a continuous $\frac{5}{16}$-in. flat deck plate, on which was laid a continuous 18 × $\frac{7}{16}$-in. track plate under each rail. Insulation was provided between the rail and the plate, and on the outside edge of the latter a single 6 × 4 × $\frac{5}{8}$-in. angle was riveted. The floor weighed 886 lb. per foot of single track.

The shallow floor used by the Chicago & Northwestern Railway for track elevation in 1899 is made with very heavy shallow transverse floorbeams about 5 ft. apart which support longi-

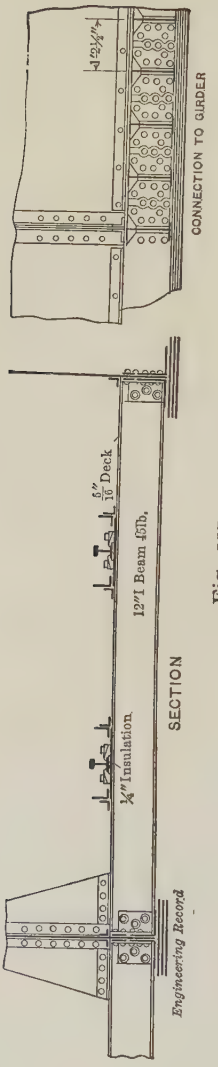


Fig. 117.

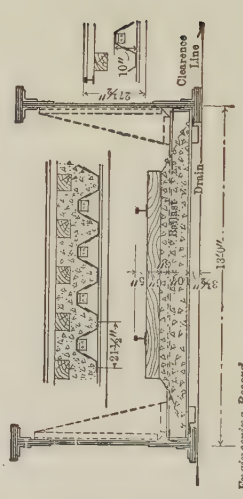


Fig. 118.

tudinal steel troughs carrying wooden sleepers under each rail. The floorbeams were made of a pair of 10-in. 25-lb. channels back to back with an $8 \times \frac{3}{4}$-in. top cover plate and a $9\frac{3}{4} \times \frac{1}{2}$-in. bottom cover plate. The ends of the floorbeams were riveted to gusset plates, which help make the connection to vertical web stiffeners in the main girders. The rail troughs were made with two $3\frac{1}{2} \times 6 \times 3\frac{1}{2}$ Z-bars, one $15\frac{1}{2} \times \frac{5}{16}$-in. bottom plate, two $3\frac{1}{2} \times 3\frac{1}{2}$-in. wide angles, and two $5 \times 3\frac{1}{2} \times \frac{1}{2}$-in. guard angles. The total weight of the floor was 696 lb. per linear foot of single track.

The structures for the 40th St. track elevation of the Chicago Junction Railway were designed by Mr. W. M. Hughes, consulting engineer, and were built and erected by the American Bridge Company. In selecting the types of shallow floor, very careful consideration was paid not only to the different constructions already described here, but to several other types which have been previously used by other engineers. Among the more important of these was a system of transverse troughs made with special U-shaped plates rolled by the Pennsylvania Steel Company, which are arranged as shown in the accompanying diagram, with their flanges turned alternately up and down and overlapping so as to be riveted together with single lines of rivets at a considerable height above the bottom of the trough. This feature makes the trough watertight, and its strength in flexure can be easily varied by rolling the trough top or bottom to any required thickness. The ends of the trough rest directly on the lower flange angles of the main girders, and the inverted sections are secured to the girder webs with bracket angles. For a trough 10 in. deep over all, the weight of the floor with ballast is 580 lb. per linear foot of single track above clearance line.

A combination of these floor troughs with a beam and stringer floor is made by using 18-in. 55-lb. transverse I-beams 6 ft. 8 in. apart, which are web-connected and kneebraced to the main girders, but are clear of their lower flanges. Longitudinal 12-in. 40-lb. I-beams are web-connected to them 5 ft. each side of the center, and support on their webs and lower flanges transverse trough sections which receive the ballast and reduce the width of the latter 3 ft. below what would be required if the troughs were connected directly to the main girders. This floor and ballast weigh 650 lb. per linear foot of single track and is $28\frac{1}{2}$ in. deep. Another variation is made by the use

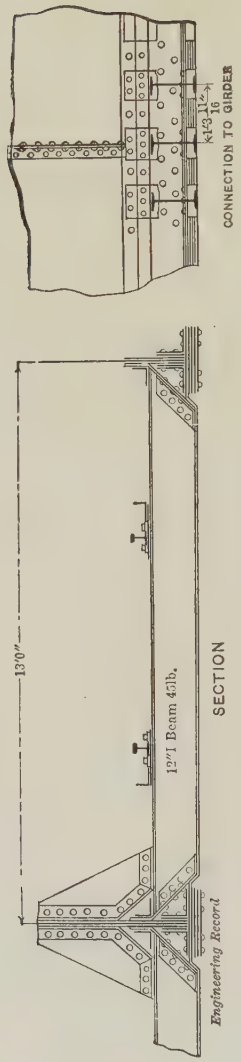


Fig. 119.

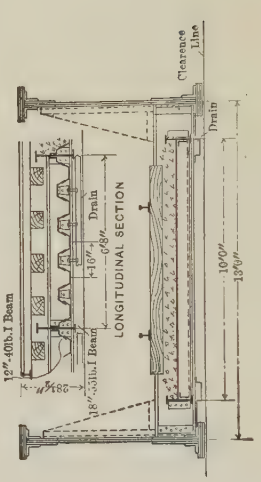


Fig. 120.

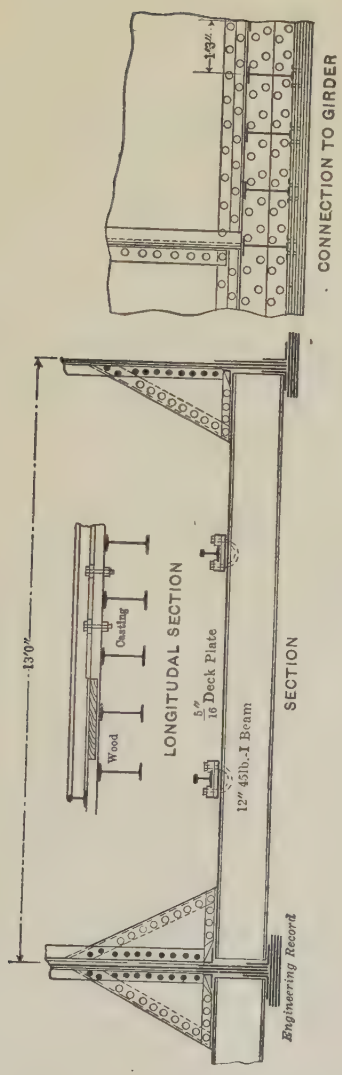


Fig. 121.

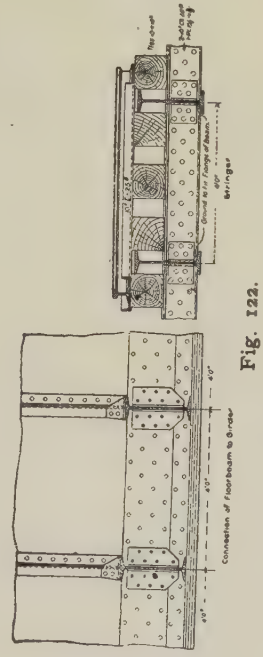


Fig. 122.

of shallow plate-girder floorbeams 8 ft. 4 in. apart, which support on their lower flanges longitudinal troughs 6 in. high

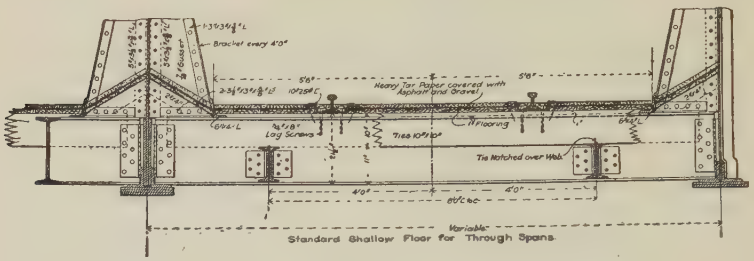

Bracket every 4'0"
Heavy Tar Paper covered with Asphalt and Gravel
Flooring
Lag Screws
Tie Notched over Web
Standard Shallow Floor for Through Spans

Fig. 123.

made with the Pennsylvania Steel Company's shapes. This floor has an extreme depth of $28\frac{3}{8}$ in., and with ballast weighs 700 lb. per linear foot of single track.

Delaware, Lackawanna & Western Railroad.

The shallow floor used for the freight track spans of the Ogden Street Bridge, Newark, is the standard construction for through plate girders with limited head room on the Delaware, Lackawanna & Western Railroad. The floorbeams have the same depth of 20 in. and are 4 ft. apart for all spans, but their weight varies according to the distance between centers of main girders, and is 65 lb. per ft. for 13 ft., 80 lb. for 14 ft., and 100 lb. for 15 ft. They are web-connected to the girders with their lower flanges bearing on the girder flange angles. The connection angles are set $\frac{1}{4}$ in. beyond the ends of the beams so that there is no danger of the beam webs touching the girders, and the connection rivets are calculated to carry all the load without allowing the flange-seat bearing to be loaded.

The stringers have an I-shaped cross-section made with two channels, back to back, with a plate riveted between their webs and projecting beyond their top flanges $\frac{1}{2}$ in. to form a rib engaging a saw-cut in the bottom of each tie and preventing the latter from slipping. Longitudinal channels are laid, flanges up, on the ties and form continuous beds for the rails and are held in position by lag screws through cast-iron caps, which secure the rails to the ties, thus serving the double purpose of continuous track plates and to prevent the longitudinal displacement of the ties. The ties extend nearly the full length of the floorbeams and are decked with 1-in. longitudinal boards

laid close, sloped up at each side at an angle of 30 degrees and covered with tar paper, asphalt, and gravel.

The standard ballast floor for deck plate girder spans on the Chicago, Milwaukee & St. Paul Railway is made with 8 × 8-in. × 14-ft. long leaf or loblolly pine ties laid close and every fourth tie bolted to the girders. Similar timbers bolted to the ends of the ties at intervals of 4 ft. retain the ballast in which the cross-ties are bedded in any required position. All timber, except track ties, is treated, after framing, with 12-lb. per cubic foot of creosote oil, and all holes bored in the field are treated with hot creosote oil. The clearance between the ends of adjacent spans and between the end of a span and the abutment wall plate is covered by a horizontal steel apron plate spiked to one tie only and free to slide back and forth at the opposite edge.

Referring to this construction, Mr. C. F. Loweth, engineer and superintendent of bridges and buildings, wrote Dec. 20, 1904:

"I am sending you a print of our plan which shows the type of ballast floor bridge we have been using for some little time past to some extent, principally where the track is curved. The advantage of this type is that common to any ballast floor bridge, that the danger of fire is eliminated. The track is maintained by the ordinary section forces, and can be shifted slightly as occasion may require, and where there is superelevation it can be varied much easier than the ordinary bridge floor, and last, but not least, it is a much safer floor. In these bridges we have protected the top flanges against rust, not only by extra precautions in painting them, but by covering them with a heavy cotton duck saturated with a preservative. The duck has been ironed on to the steel surfaces with hot irons so that it adheres tightly. With properly creosoted timber we expect twenty years' life, and probably will get considerable more."

Standard spans up to 30 ft. which require a very shallow floor are made as shown in the part cross-section of type E (see Fig. 62) with simple through girders without flange cover plates. Two girders about 3 ft. apart are provided for each rail and carry it on 8 × 12-inch ties 13 in. apart on centers, which have their ends fitted to bear against the girder webs. The ties rest on continuous 3 × 4-in. shelf angles riveted to the girder webs, with their horizontal flanges reinforced by vertical web-stiffener angles about 2 ft. apart. Guard timbers 4 × 5 in. are fitted

between the ties and the top flanges of the girders and are toe-nailed to the former and spiked to the wall plates at abutments and piers. There is no lateral or sway bracing for spans of 25 ft. or less, and the girders for each rail are connected only by the track ties and bolts. The adjacent girders of the two pairs are spaced about 1 ft. 8½ in. apart and are connected by vertical transverse diaphragm plates riveted to the web-stiffener angles about 6 ft. apart. Nailing strips are bolted to the upper edges of the diaphragms, and to them are spiked the center 2 × 6-in. timber.

Where the extreme minimum of depth is necessary in shallow floors they are sometimes made, up to 12-ft. spans, with the 8 × 12-in. ties seated on the horizontal legs of the lower flanges of two plate girders for each rail. The timbers are arranged as in type E, and the girders are seated on cast-iron pedestals with an H-shaped cross-section.

CHAPTER X.

CONCRETE AND CONCRETE-STEEL FLOORS.

In the Bay Ridge Improvement of the Long Island Railroad, and in the track elevation of the Manhattan Beach Railway, there are approximately 60 bridges, comprising both railroad structures and overhead highway bridges. Deck bridges are used wherever there is sufficient headroom.

In narrow streets, or where the crossings are at right angles, the bridges are made with single spans up to about 60 ft. in length, in other places the girders are supported on columns just back of the curb lines, thus dividing the space into three spans.

The track is laid over all bridges in stone ballast, which provides for the continuity of the roadbed from the embankment across the abutments and greatly promotes satisfactory maintenance. The ballast is in all cases supported by solid floors of reinforced concrete, carried by steel beams and girders arranged so as to limit the spans of the concrete to a maximum of about 5 ft., the minimum thickness of the concrete being 8 in.

Preliminary studies were made developing several comparative designs for floor construction; from these the most satisfactory features have been combined to secure three standard types of bridges to which all of this construction substantially conforms, namely, railroad deck spans, railroad through spans, and highway spans.

Among the most important considerations determining the selection of the concrete floor system were: the diminution of impact effects by the increased inertia; the reduction of maintenance charges; and the protection of streets from drip, dirt, etc. It is true that the adoption of ballasted concrete floors increases the dead load transmitted to the main girders to a total three or four times that due to ordinary cross-ties laid on open stringers. Notwithstanding this increase, however, the main girders have been designed with the same sectional areas as would have been provided for wooden floors. The idea of this construction is that the inertia of the concrete and ballast reduces the effect of impact more than enough to compensate for the added dead load.

The fundamental consideration in the design of the concrete floor was to make a composite construction in which the reinforced concrete and the principal steel members would be thoroughly bonded together so that the reinforced concrete shall develop suitable beam and arch action, thus utilizing its own strength in transferring the live load to the steel members, which it also protects and bonds together. Special provisions are also made for very thorough waterproofing and drainage.

The construction was designed in accordance with the Long Island Railroad Co.'s general specifications for steel structures, of 1905, as regards loads, impact, etc., but the assumption was made that the track ballast and concrete would distribute the

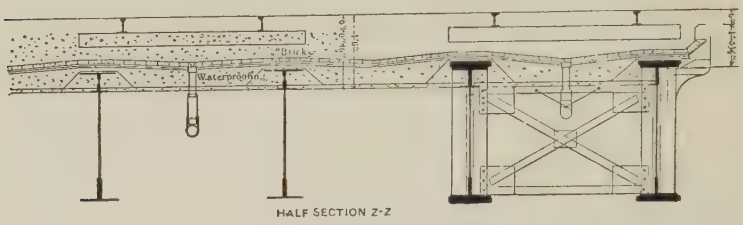


Fig. 124.

driver loads (Cooper's E-50) over the length of the wooden ties transversely and from center to center of drivers, 5 ft., longitudinally, and when it is understood that the impact allowance for the floorbeams is 100 per cent of the live load and that after the concrete is properly reinforced, and bonded to the steel work, it will materially stiffen and strengthen it, the above assumption appears to amply satisfy all considerations of strength and safety.

The 1 : 2 : 4 concrete is made with Portland cement and $\frac{3}{4}$-in. broken stone reinforced with twisted steel bars from $\frac{1}{4}$-in to $\frac{5}{8}$-in. square, the smaller ones being laid transverse to the lines of stress in order to prevent danger of cracking. The reinforcement is proportioned according to Thacher's formulæ which give about 1 per cent of steel for the cross-sectional area of the slab. It is required that the steel must be free from paint, grease, or dirt, when bedded in the concrete. The total thickness of the floor in deck spans averages about 2 ft. and in through spans about 3 ft. Train loads are under no circumstances imposed on the concrete until it is at least ten days old, and in many cases much older. It is expected that the first cost of this type of

floor may be easily 50 per cent more than that of the old style floor, with open steel stringers, but it is probable that considering the cost of maintenance, the capitalization will be little, if any, greater, and that much more satisfactory traffic conditions will be secured.

The Rockaway Avenue Bridge affords an illustration of the typical short-span deck structure for skew crossings. It carries four tracks across the street and sidewalks with a vertical clearance of 14 ft. There are eight 66-ft. plate girders $6\frac{1}{2}$ ft. deep

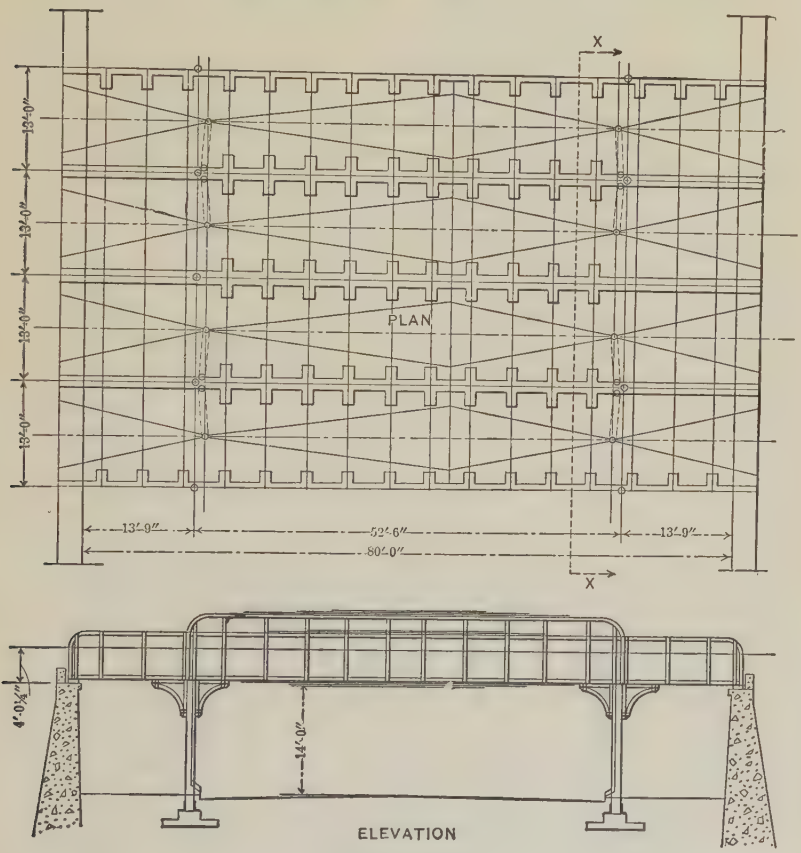


Fig. 125.

braced together in pairs, one for each track, with ordinary sway frames. The web plate, $\frac{1}{2}$ in. thick, is made in two half length pieces, and the $8 \times 8 \times \frac{5}{8}$-in. flange angles and the 19-in. cover plates are made in single unspliced lengths. The ends of the

lower flanges have sole plates bearing on riveted pedestals about 9 in. high, which give 24 × 28-in. bearings on the masonry. Horizontal $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$-in. shelf angles with their vertical flanges up, are riveted to both sides of the girder webs from end to end, with their horizontal flanges flush with the lower edges of the main flange angles.

These shelves receive the continuous concrete floor slabs which inclose the top flanges of the girders, above which they are about 2 in. thick. The upper surfaces of the slabs are pitched to longitudinal lines between the centers of the girders where they have a minimum depth of 8 in. They are reinforced by $\frac{5}{8}$-in. transverse rods 6 in. apart, every alternate rod being made continuous across the full width of the bridge by 2-ft. laps, splicing them over the top flanges of the girders where they are bent upwards as shown in the accompanying cross-section. The other transverse rods reach only from girder to girder and are not bent upwards at the ends. The $\frac{1}{4}$-in. longitudinal rods are spaced 12 in. apart and are lapped 12 in. for splices. The concrete is continued 2 ft. beyond the center of each outside girder and is carried up $17\frac{1}{2}$ in. above the top flange angels to form a low parapet or fascia girder which is reinforced by the projecting ends of the transverse rods and serves to retain the ballast.

The upper surface of the concrete is thoroughly waterproofed with a $\frac{1}{2}$-in. thickness of felt and asphalt on which ordinary red brick are laid flat on a $\frac{1}{2}$-in. mortar bed so as to protect the waterproofing from accidental blows of pick or bar when the trackmen adjust the ballast. The drainage from the center gutters is delivered through short 2-in. vertical cast-iron pipes to lines of 4-in. wrought iron pipes pitched both ways from the center of the span to the back walls of the abutments through which they pass and deliver to the sewers behind them.

A typical four-track three-span through bridge is illustrated by the Remsen Avenue crossing, 80 ft. long between faces of abutments. Two continuous concrete footings $52\frac{1}{2}$ ft. apart on centers and 65 ft. long are built in the curb lines.

On each footing are seated five vertical columns, and to the tops are riveted solid web kneebrace brackets, with cap plates 14 in. wide which are continuous across the column tops and receive the lower flanges of the girders. The floorbeams are 24-in. 80-lb. I-beams spaced 5 ft. apart, and web-connected to the main girders with their lower flanges flush with the second cover plates of the latter. The upper flanges are field-riveted

to kneebrace gusset plates having curved flange angles extending to the top flanges of the main girders.

The floorbeams are protected by concrete in which they are embedded, with the exception of the lower surfaces of their bottom flanges, which are flush with the bottom of the concrete. The concrete around the beams is continuous with that of the floor slabs between them, and is joined to it with curved fillets which give an arch-like appearance to it in longitudinal section. The slabs are 11 in. thick, and have the upper surfaces pitched to drain pipes in the middle of the center line of each track.

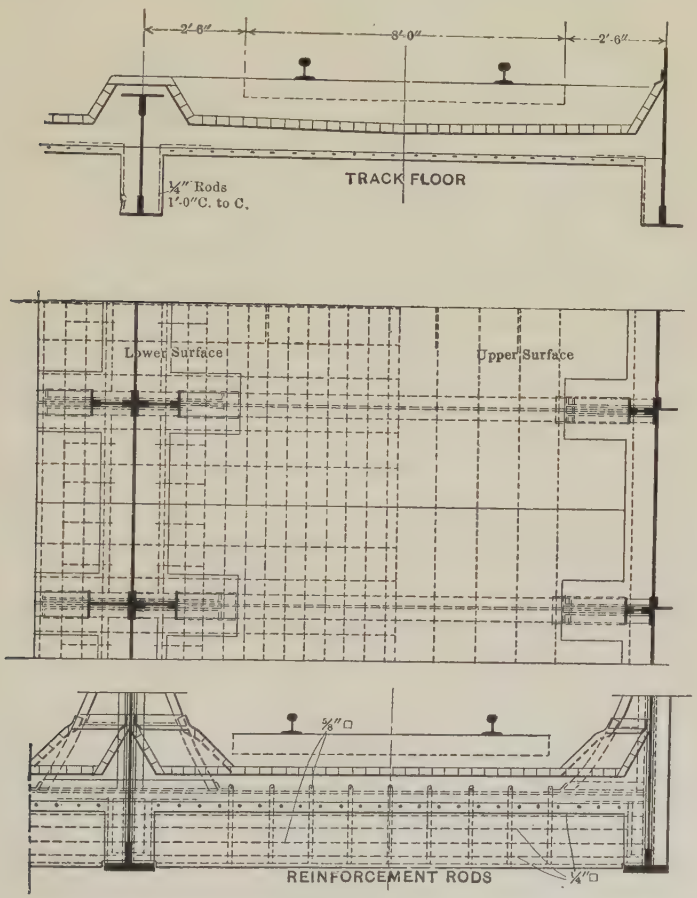


Fig. 126.

The concrete is reinforced with $\frac{5}{8}$-in. longitudinal bars, 12 in. apart in the upper surface and 6 in. apart in the lower surface.

These bars pass through holes punched in the webs of the floorbeams, and are made continuous from end to end of the slab by overlapping 2 ft. at each joint. Transverse $\frac{1}{4}$-in. rods, 12 in. apart, are bedded near the lower surface of the concrete. Special reinforcement is provided for the concrete inclosing the floorbeams by transverse $\frac{5}{8}$-in. rods and vertical $\frac{1}{4}$-in. U-bars. At each end of the bridge the slab projects over the top of the back wall of the abutment and rests on it with a sliding expansion joint. The upper surface of the concrete is covered with a $\frac{1}{2}$-in. coat of waterproofing and a single course of red bricks laid flat in cement mortar and grouted. Above this there is stone ballast, 4 in. deep, to the lower side of the ties, 6 in. thick.

At the longitudinal edges of the slabs the concrete is carried up at an angle of about 30 degrees to the base of the rail, the waterproofing and brick protection being made to correspond. This flashing is carried around the gusset plates at each floorbeam, and is very carefully molded to them so as to insure complete protection of the structure from the ballast and prevent any leakage of rain-water between the concrete and the steel work. Considerable difficulty has been encountered in securing a suitable finish at the upper edge of the flashing and in protecting the joint there so as to thoroughly exclude the water.

This desired arrangement has finally been attempted, as shown in the details, by riveting to the web of the main girder and that of the gusset plates a light horizontal angle with the horizontal flange bent downwards, forming an oblique section and engaging the upper edge of the brick flashing. On the inclined flange of the gusset plate, the bent angle is replaced by a short curved section of 10-in. channel.

The floor construction in the side spans is modified slightly as shown in the cross-section so as to carry the concrete, waterproofing and brick course over the top flanges of the 36-in. girders.

A modification of the Remsen Avenue crossing is illustrated in the Avenue G crossing, which is a five-track bridge on a very sharp skew. The perpendicular distance between the faces of the abutments is 64 ft., while the total length of the bridge is about 255 ft. There are six lines of girders 13 ft. apart on centers, each line consisting of an 89-ft. 10-in. center span and two 19-ft. 8-in. end spans, supported by the abutment walls and by columns as in the Remsen Avenue bridge, but the floor system varies from the latter in that it is made with 12-in. I-beams,

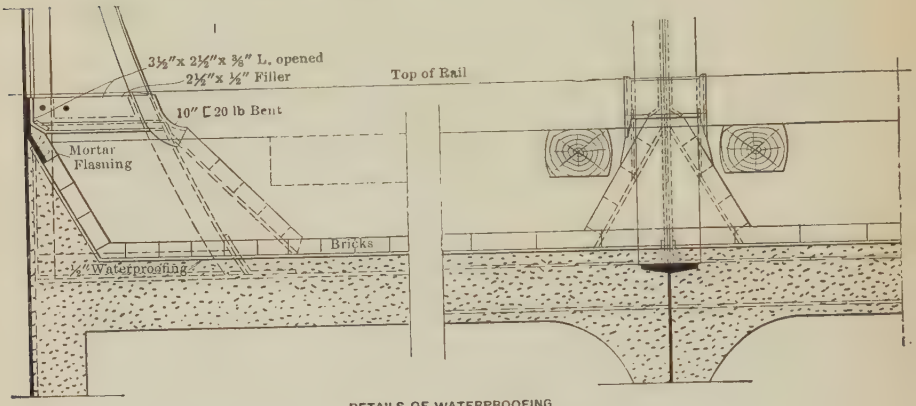
12 in. apart. Every sixth I-beam weighs 40 lb. per foot, and is field-riveted to gusset plates extending to the top flange of the main girder. The intermediate beams weigh $31\frac{1}{2}$ lb. and are connected to the girder web with pairs of flange angles. All of the floorbeams are notched at the ends to clear the lower flanges of the girder. The concrete slabs have a minimum thickness of $13\frac{1}{2}$ in. at the center line of each track, and from this point the upper surface slopes upwards nearly to the webs of the main girders where it joins the flashing. The concrete is not reinforced, and the distance between the I-beams is so small that the latter take the loads directly, and the concrete serves only to protect them and provide a continuous surface retaining the ballast.

Solid-floor bridges for short spans on the Baltimore & Ohio Railroad have been designed of several types in which the loads are chiefly carried by steel beams and girders, and the latter are entirely inclosed in concrete integral with the floor slab, which at the same time affords protection against rust; provides a continuous platform for the ballast so that the track can be laid like that on the adjacent fills, and may be shifted to different positions within the limits of the platform; and promotes the economy of maintenance.

The shortest openings, namely those for culverts, are covered with simple flat slabs of reinforced concrete 12 in. thick, which have bearings across the full width of the side walls 3 ft. thick. They have clear spans varying by 1-ft. increments from 5 ft. to 12 ft., and are proportioned for a live load consisting of a Cooper's E-50, $177\frac{1}{2}$-ton engine and tender, with an allowance of 50 per cent for impact. The dead load, exclusive of the concrete, which is assumed to be self-sustaining, is assumed to be 2,000 lb. per lin. ft. of track. The consideration of it is, however, immaterial, since the maximum stresses are determined from the axle loads.

The concrete is made 1 : 3 : 5 with Portland cement and $1\frac{1}{2}$-in. broken stone well rammed. It has a uniform thickness of 12 in. except at the edges, where the distance is increased 6 in. to make a fascia girder or parapet 12 in. wide to retain the ballast. Under each rail there are embedded in the concrete a number of steel rails set close together and alternating with reversed rails closely packed between them. Between these groups and on either side of them are single rails spaced uniformly for a distance of 5 ft. each side of the track center. On the edges of

the bridge, and between tracks of a double-track bridge, the spaces beyond these limits are occupied by additional rails uniformly spaced a little farther apart. The rails have at each end



DETAILS OF WATERPROOFING
PART TRANSVERSE SECTION OF FLOOR

Fig. 127.

direct bearings 18 in. long on the tops of the side walls. The spaces between the webs of rails set close together are filled with 1 : 3 Portland cement mortar. It is not required that the rails shall be new, but they must be of good quality, and free from cracks or serious defects, and of the full weight required in the table.

A convenient table gives the different numbers of 60, 70, and 80-lb. rails required for single- and double-track bridges. A single-track 5-ft. span requires 24 60-lb. rails, and the estimated cost is $106, exclusive of the side walls, and allowing $39 for the concrete and $9 for the roadbed. If made with 80-lb. rails the same number are used, and the cost is increased to $125. Double-track bridges of the same span require 45 rails. For 12-ft. spans 80-lb. rails only are specified, and 46 of them are required, corresponding to a total cost of $352, allowing $14 for roadbed and $62 for concrete. Single-track bridges with clear spans of 6, 7, 8, 9, and 10 ft. each, require 20 60-lb. rails and from 16 to 20, 70 or 80-lb. rails. The estimates of cost are based on a price of $1\frac{1}{2}$c. per lb. for the rails, $\frac{1}{2}$c. per lb. for cutting and placing, $4.00 per cu. yd. for materials and $2.50 per cu. yd. for labor in concreting, 60c. for materials and 20c. for materials per lin. ft. of track for roadbed.

For spans varying from 12 to 26 ft. in the clear a standard construction has been designed for the loading already specified, which consists of a set of longitudinal I-beams, uniformly spaced across the full width of the bridge, which have at each end over 2 ft. of direct bearing on the side walls, and are entirely inclosed in a solid mass of concrete projecting 2 in. below the lower flanges, and from 4 to 5 in. above the top flanges. In this case the concrete serves chiefly to protect the beams and secure a continuity of surface, the loads being directly carried by the girders, which are proportioned for unit stresses of 16,000 lb.

The I-beams, thoroughly cleaned and free from rust, paint, grease, and dirt, are strongly clamped in the required position, and 1 : 2 : 4 concrete made with 1-in. broken stone is thoroughly rammed between them. The bottom, sides, and upper surfaces are finished with a ½-in. coating of 1 : 3 mortar, rammed in with the concrete and not plastered on. After the forms are removed the surface of the concrete is washed with cement grout to fill the pores and give uniformity of color.

When the bridge is constructed under traffic the track is supported above it in such a way as to relieve the I-beams and concrete of all loads and impact until the concrete is satisfactorily set. When the platform is built on false-work parallel to the track, and afterwards moved transversely to the bridge seats, the latter are immediately grouted so as to secure even bearing throughout. The upper surface of the concrete is crowned so as to provide for drainage to gutters adjacent to the fascia girder and, in double-track bridges, to center gutters, all of which are provided at frequent intervals with 4-in. vertical drain pipes.

A 12-ft. single-track span requires thirteen 12-in. I-beams weighing 7,012 lb., 16.6 cu. yd. of concrete, and costs $312.00, allowing $159.00 for the beams erected, $108.00 for the concrete, and $13.00 for the roadbed. This corresponds to a total cost of $202 for material and $110 for labor. A 26-ft. span requires eleven 24-in. 80-lb. I-beams weighing 26,980 lb., and 49 cu. yd. of concrete, and costs $936, of which $563 is for steel, $319 for concrete, and $24 for roadbed. This corresponds to $661 for material and $275 for labor. Double-track bridges require two less than double the number of beams required for single-track bridges, and are estimated to cost 1.9 times as much as single-track bridges of the same span. Where

old I-beams are retained in position in the new bridges they are credited at the rate of $1 per lin. ft. each. Where they are removed they are credited at 50 c. per lin. ft.

Short spans of more than 26 ft. are usually made with plate girders having special steel and concrete floor platforms. The character of the deck spans is clearly shown by the design for bridge 34 A, of the Washington branch, which is 78 ft. long over all and 72 ft. 8 in. long between abutments. It is propor-

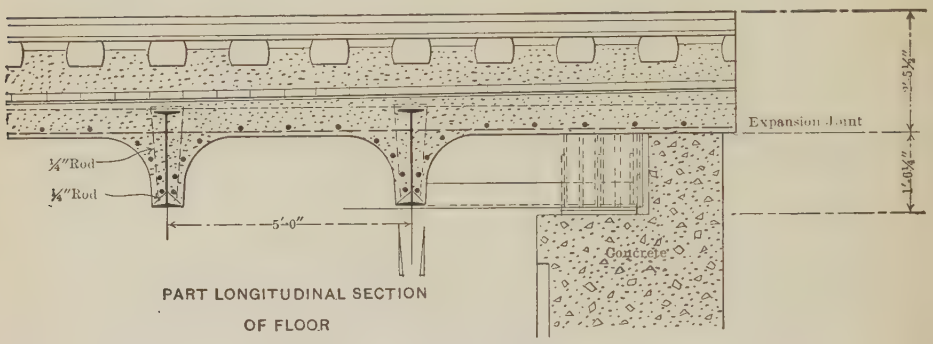


Fig. 128.

tioned for the loading already mentioned, which gives a maximum shear of 320,200 lb., and at 10,000 lb. per sq. in. requires 32 in. of net cross-section in the web, which is made with a $95\frac{1}{2} \times \frac{1}{2}$-in. plate. The maximum moment is 5,506,500 ft.-lb., which, divided by 8 ft., gives a flange stress of 688,300 lb. and requires a net area of 43 sq. in. at 16,000 lb. per sq. in. The flange is made up with two $8 \times 8 \times \frac{5}{8}$-in. angles and a maximum of three $18 \times \frac{1}{2}$-in. cover plates, which, together with $\frac{1}{8}$ of the web plate, gives a total net section of 46.7 sq. in. The top flange cover plates are all of them full-length plates, so as to provide a continuous bearing at uniform level for the floor troughs. The girders are seated on standard cast-iron pedestals, and are connected together at web-stiffener angles with ordinary vertical transverse sway bracing.

The floor consists of rectangular transverse troughs filled with concrete, above which there are 15 in. of broken stone ballast. The floor troughs are made in single lengths projecting $3\frac{3}{4}$ ft. beyond the outside girders, and having their ends closed by 15-in. channels forming fascia girders to retain the concrete. The troughs are 8 in. deep, and are made with $12 \times \frac{3}{8}$-in. top and

bottom plates connected with $5 \times 3\frac{1}{2} \times \frac{3}{8}$-in. angles having their vertical flanges riveted together so as to dispense with side plates. The trough section has a moment of inertia $I = 60$, 59 being required by the formula

$$I = \frac{24{,}100 \times 6.5 \times 4 \times 12}{8 \times 16{,}000}.$$

The 44-ft. 9-in. span of the undergrade crossing of the Western Maryland Tide Water Railroad on the Montclair branch of

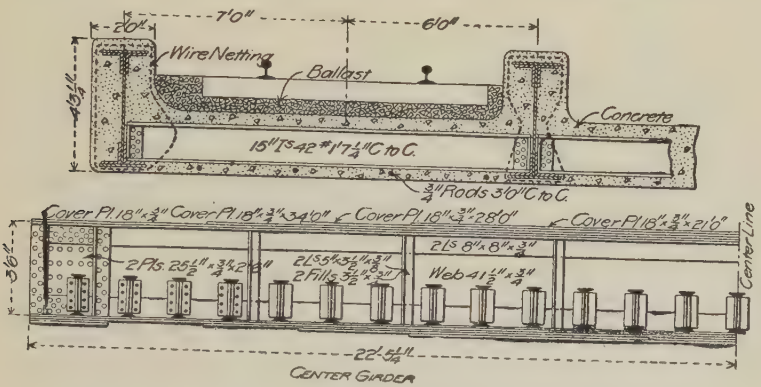


Fig. 129.

the Baltimore division is typical of the solid-floor type of skew through double-track plate-girder spans. Here the massive

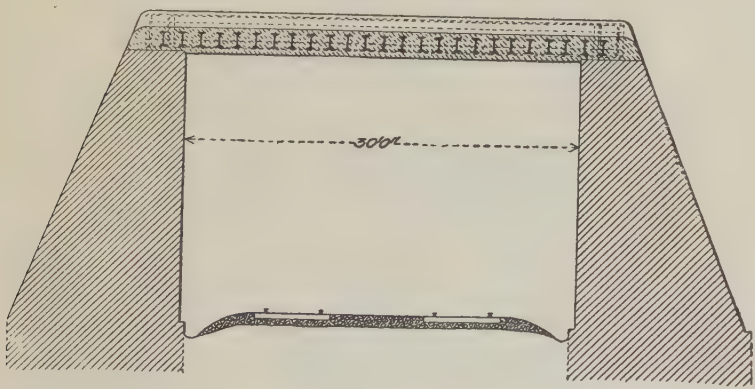


Fig. 130.

concrete abutments are skewed $53\frac{1}{2}$ degrees with the track, and serve as retaining walls for the deep fill. They carry three plate girders, which, together with the floorbeams, are entirely

inclosed in a monolithic mass of concrete. The girders are wrapped with galvanized wire netting of 12-in. mesh, and the

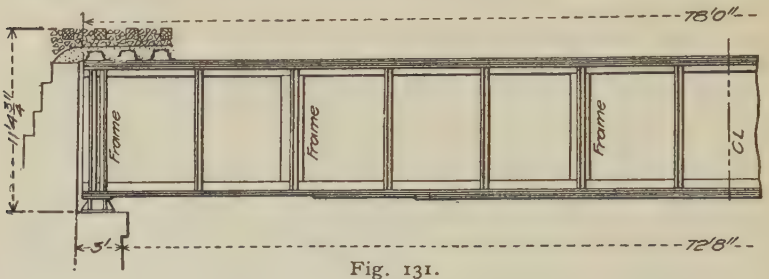


Fig. 131.

15-in. floorbeams, 19¼ in. apart, are bedded in concrete 25 in thick, which forms a thick slab extending beyond the top and

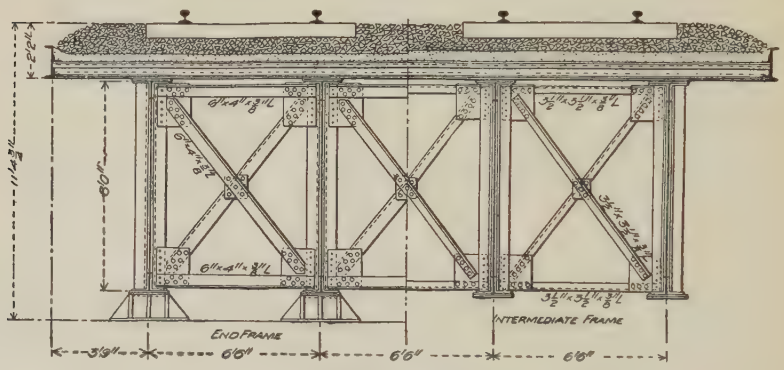


Fig. 132.

bottom flanges. Under the lower flanges it is reinforced with ¾-in. longitudinal rods 3 ft. apart. The upper surface of the

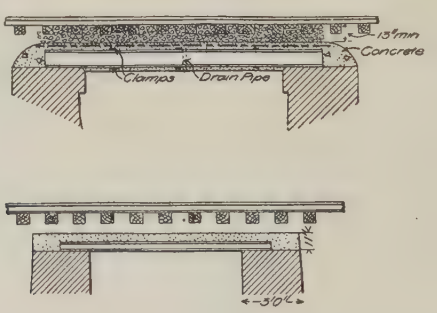


Fig. 133.

floor slab is connected with the casing of the main girders, and receives stone ballast for the track. The concrete is made

1 : 3 : 5 with $1\frac{1}{2}$-in. broken stone between the beams and $\frac{3}{4}$-in. broken stone on the top, bottom, and sides.

A large number of Erie main line, yard and local tracks substantially parallel to each other and as close together as possible

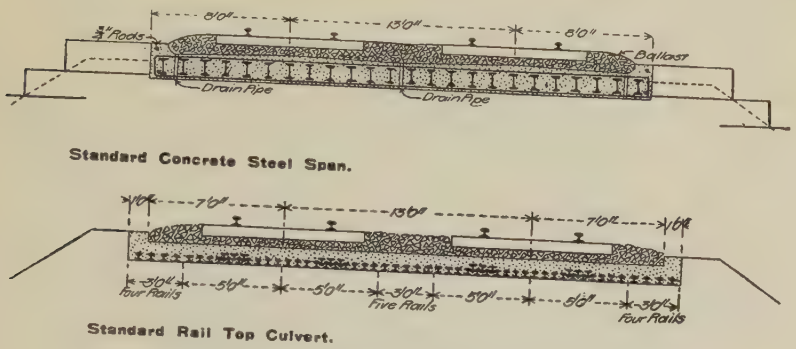


Fig. 134.

are concentrated at 51st Street, Chicago, and cross it nearly at right angles with a vertical clearance of about 16 ft. above the surface of the street. The tracks are carried on fills on both sides of the street, which is 43 ft. wide between curbs, but over

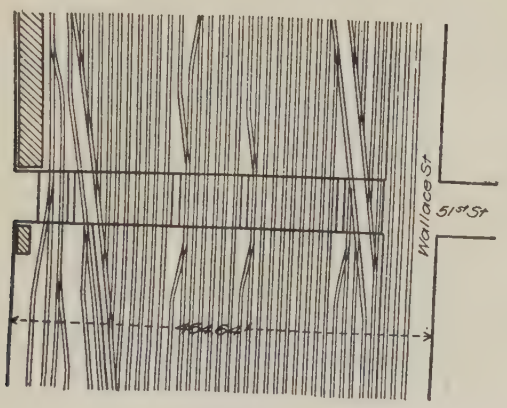


Fig. 135.

it they are carried on a deck bridge which consists of a platform 67 ft. wide and 433 ft. long, parallel with the axis of the street. The platform has three lines of longitudinal plate girders parallel with the axis of the street, which are supported on one row of columns in the center of the streets and on one row at each curb.

The girders carry across their top flanges 20-in. 80-lb. I-beams 15 in. apart on centers, and about 46½ ft. long. Both ends of

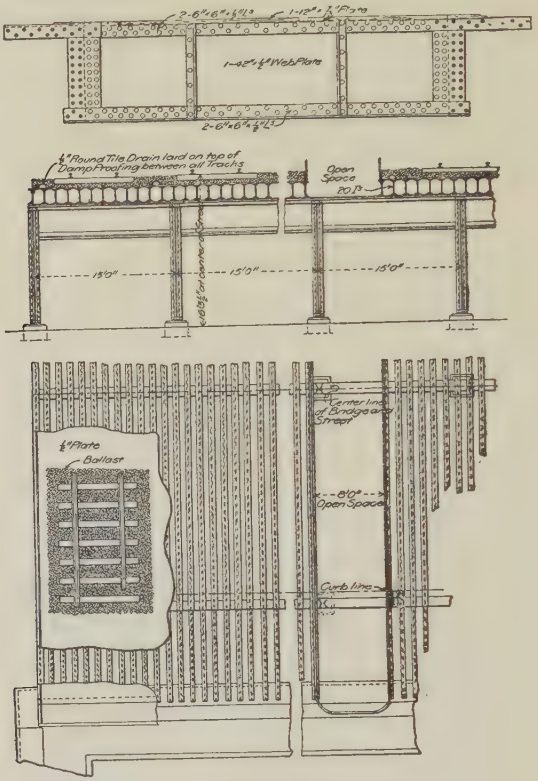


Fig. 136.

these beams are extended by 12-in. I-beams supported at the outer ends on a retaining wall at each side of the street. The

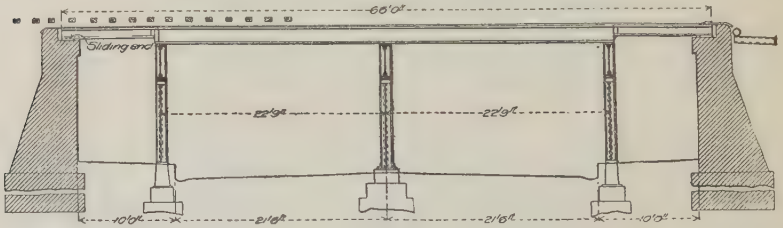


Fig. 137.

top flanges of the beams are covered with a continuous horizontal plate ½ in. thick, countersunk-riveted to them. The outer

edges of the bridge, transverse to the axis of the bridge, are provided with a fascia girder 30 in. deep, which confines 10 in. of sand ballast spread over the surface of the platform. In this ballast the ties are bedded so that the tracks can be arranged in any position desired. As the street underneath the tracks is inclosed by solid walls on both sides and by the solid floor overhead, it presents a tunnel-like appearance and is very dark. Some light, however, is admitted through four wells 8 ft. wide which extend across the full width of the bridge, and are inclosed by fascia girders and gas-pipe hand rails similar to those on the outer sides of the platform.

The transverse I-beams are connected to the longitudinal girders by two field-rivets through the adjacent flanges at each intersection. Generally they were shipped in groups of three, shop-riveted to the floor plates $44\frac{1}{2}$ in. wide and 46 ft. $10\frac{1}{2}$ in. long. The rivets are pitched 12 in. apart in each flange, staggered, and are countersunk in the $\frac{1}{2}$-in. deck plate. The transverse edges of the plates are butt-jointed with $\frac{1}{2}$-in. clearance at the center line of every third beam, where one row of holes was left open for field rivets. Pairs of vertical web-connection angles are riveted to the ends of the I-beams, and their outstanding flanges are connected by single horizontal 6 × 4-in. angles 3 ft. $2\frac{3}{4}$ in. long, which are shop-riveted to them. These angles serve as shelf supports for the 12-in. I-beams 9 ft. $8\frac{3}{4}$ in. long, which carry the deck plates across the sidewalk. These I-beams have web-connection angles field-riveted to those on the 20-in. I-beams, and were shop-rveted to the deck plates in groups of three like the former. The deck plates project 6 in. beyond the ends of the beams to form sliding aprons resting on the tops of the back walls of the abutments. The lower flanges have shoe plates countersunk-riveted to them, which rest on the abutment walls and are beveled to correspond with the inclination given to the beams to provide for drainage. The total steel weight of the bridge is 991,801 lb.

Great pains were taken to provide a thorough waterproofing for this bridge so as to prevent any danger of leakage or dripping upon the street below. The upper surface of the floor and the inner sides of the fascia girders were thoroughly cleaned with a torch just before the application of the waterproofing. Then hot Hydrex compound was spread over it with a mop, covering about one square yard with each application. The Hydrex felt, in rolls 36 in. wide, was applied to the mopped surface in courses

laid transverse to the axis of the bridge, reaching continuously across the full width of the floor and extending up vertically 4 in. on the webs of the fascia girders. At the end of the bridge the felt was applied in four-ply layers. Successive applications were made with the edge of the felt moved back 18 in. so as to give an overlap of 2 in., thus presenting a sort of channeled surface and maintaining everywhere a thickness of four ply, each ply lapping those below it 18 in. Each ply was laid in the hot compound thoroughly mopped over the preceding layer, and was well rolled while hot. The longitudinal joints made by the ends of the successive rolls of felt were lapped 12 in. The sides of the fascia girder were counterflashed with two ply of felt laid in hot compound and extending from a point above the first flashing to a distance of 6 in. beyond the fascia girder on the floor of the bridge. Special care was taken to cement the flashing thoroughly, and after it was completed the surface of all the flashing and floor was thoroughly covered with hot compound and protected by old bricks laid side by side. The operations of this process were rigidly specified and very carefully carried out, the result being that the work is considered thoroughly waterproofed and satisfactory by the contractors and engineers.

The Market Street Elevated Railway, Philadelphia, has, up to the city limits, a solid floor of special construction made with longitudinal troughs (see pp.) built with 5 $\frac{1}{16} \times 3\ \frac{5}{16} \times \frac{3}{8}$-in. Z-bars, having their horizontal flanges connected by 15 $\times \frac{3}{8}$-in. flat top plates and 17 $\times \frac{3}{8}$-in. dished bottom plates. The troughs were shop-riveted complete in sections about 7 ft. wide and 20 ft. long, the dished bottom plates being cut to clear the flanges of the transverse girders. The top plates of the troughs are spliced with short horizontal cover plates, and at the ends of the bottom plates short angles are riveted to the edges of the transverse girder flange angles to prevent any drip from following the surface of the girder. The troughs are filled with 1 : 3 : 6 concrete made with Vulcanite Portland cement filled in to a minimum height of 4 in. above the top plates alongside of the middle longitudinal drainage gutter, from whence the surface is placed on a 3 per cent grade to a curb, or fascia channel, which on the inner or track side is also covered with concrete, and is placed to the side of the longitudinal girder. To prevent shrinkage cracks, the concrete is reinforced with $\frac{3}{8}$-in. Johnson bars, 12 in. apart transversely and 18 in. apart longitudinally. In order to secure the concrete and key it thoroughly to the

vertical face of the fascia web and at the expansion joints where the concrete is retained by a similar cross-curb or fascia, horizontal bolts are made to project a few inches from the webs of the curbs or fascia, and are united by zigzag lines of wire bedded in the concrete.

The concrete is finished with 1 in. of granolithic, pitched to the gutter on the center line of the structure, the latter having a minimum grade of 1 per cent to drainage holes at both ends of every span. Stone ballast is filled in on the concrete surface, and receives the cross-ties as indicated in the detail, and the rain-water, drip, etc., drains to the open gutter and thence through short vertical pipes to inclined steel troughs with their lower ends discharging into cast-iron catch basins placed inside the main vertical columns. These catch basins waste through vertical pipes carried down inside the columns to the street level, where they discharge to underground drains connected to the sewer.

Under date of May 28, 1906, Mr. W. M. Hughes writes: "Since these plans were sent to you we have decided to put

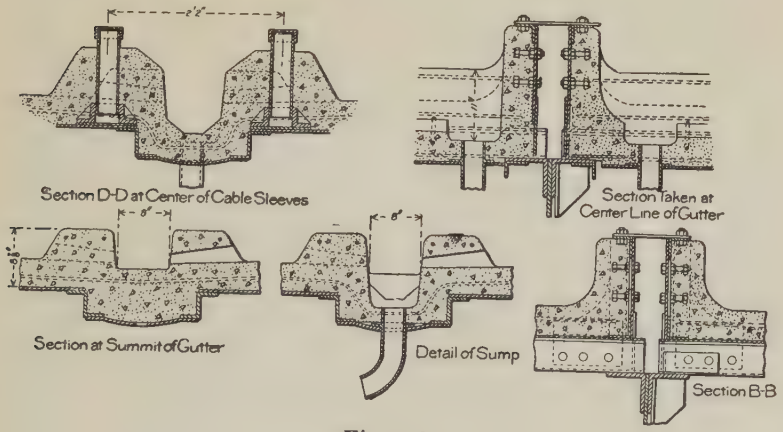

Section D-D at Center of Cable Sleeves
Section Taken at Center Line of Gutter
Section at Summit of Gutter
Detail of Sump
Section B-B

Fig. 138.

in reinforced concrete floors where ballast is used. This floor will be waterproofed with asphalt. The Chicago & Western Indiana adopted this plan, and it has proved quite successful. The shallow floor without ballast has been almost entirely abandoned by most of the roads here for track-elevation work, and wherever columns are allowed in the center of the street, deck structures similar to that at Union Avenue are the more

desirable on account of no interference with cross-overs and turnouts. In this 40th Street work there are a total of 29 street and alley crossings amounting in the aggregate to about 5,300 tons. These structures were all designed for a live load of 5,000 lb. per linear foot of track plus 50,000 lb. concentrated."

The design for the reinforced concrete floor referred to by Mr. Hughes, consists of a continuous floor slab supported on the top flanges of the floorbeams, and is of two types, corresponding to the transverse and longitudinal arrangements of the latter. One type is used with transverse floorbeams, and consists of a slab between each pair of girders with its upper surface pitched from both the longitudinal edges to a gutter under the center of the track where its thickness varies from $3\frac{1}{2}$ to 5 in. At the edge the concrete is thick enough to reach just above the rail base, and thence slopes at an angle of about 45 degrees to the edge of the gusset plates where the normal slab has a thickness of $4\frac{1}{2}$ to $6\frac{1}{2}$ in. A small triangular depression is made in the upper surface of the concrete adjacent to the girder web to retain asphalt, with which the joint is sealed to prevent percolation of water along the face of the steel. The remainder of the concrete surface is covered with asphalt waterproofing, and on it is laid stone ballast about 6 in. thick under the ties. The 1 : 2 : 4 Portland cement concrete is reinforced with $\frac{1}{2}$-in. corrugated bars in staggered rows near the upper and lower surfaces. Care is taken to mould the concrete symmetrically around the bases of the gusset plates and at the abutments. The concrete slab extends 2 or 3 ft. beyond the last floorbeam, and takes bearing on the back wall with double thickness of tarred felt.

Where the floorbeams are longitudinal the floor slab has the same general outline in cross-section as already described, except that on the outer edges the upper surface of the concrete is sloped up flush with the top flange of the fascia girders, and there are no gusset plates to interrupt its continuity. Here also the reinforcement rods are transverse instead of longitudinal, but are located, spaced and staggered in substantially the same way. At the end of the span where the concrete slab projects beyond the ends of the floorbeams to rest on the back wall of the abutment the cantilever portion is reinforced by short longitudinal bars.

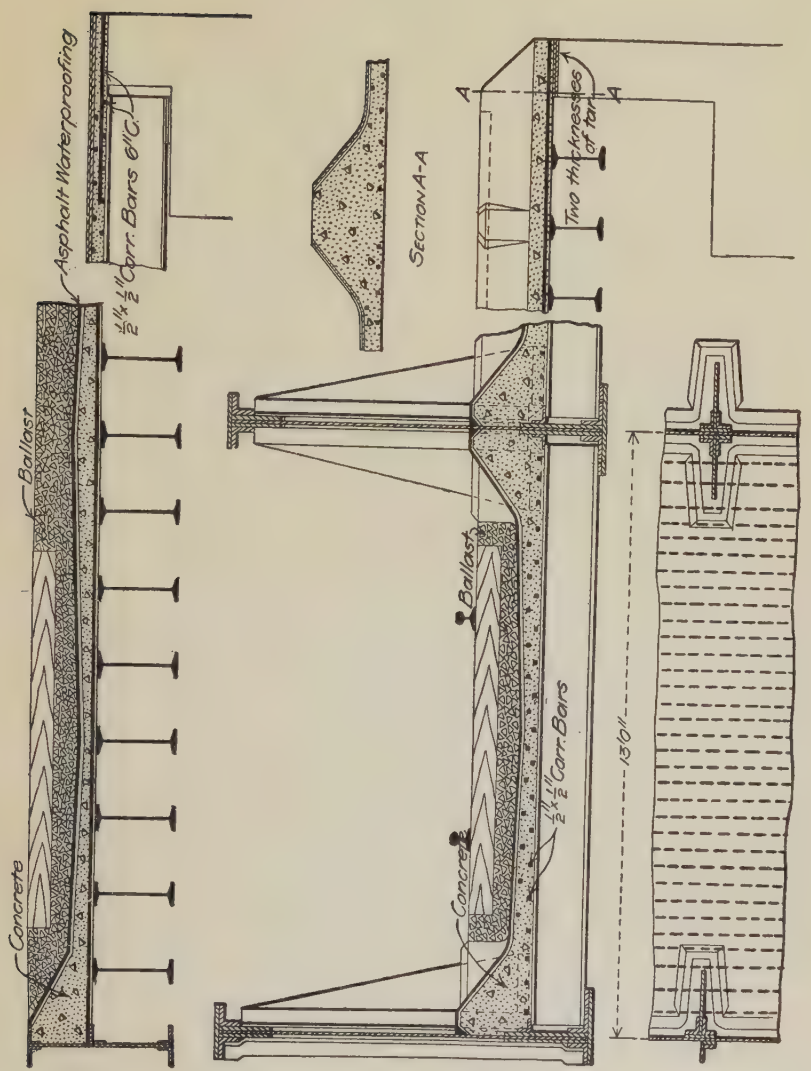

Asphalt Waterproofing
Ballast
Concrete
½"x 1½" Corr. Bars 6" C.
Section A-A
A
A
Two thicknesses of tar
Ballast
Concrete
½"x 1½" Corr. Bars
130"

Fig. 139.

PART IV

DISCUSSIONS OF STANDARD DESIGN BY BRIDGE ENGINEERS

PART IV.

DISCUSSIONS OF STANDARD DESIGN BY BRIDGE ENGINEERS

CHAPTER XI.

BRIDGE ENGINEERS FOR RAILROAD BRIDGES.

By Mr J. B. French, Bridge Engineer, Long Island Railroad.

The proper design of plate girders, as of every other class of structures, must be based on an intelligent comprehension, not only of all the considerations relating to the manufacture of the steel work in the mills and shops, and the transportation and erection of the finished parts; but also of the considerations governing the relation of the structures to the masonry and permanent track of the railroad, and to all the local conditions affecting their safety and the cost of maintenance.

The first class of considerations can receive careful and adequate attention in the designing room of any first-class bridge company, but the latter can only be fully known and appreciated by those familiar with all the local conditions and sufficiently experienced in railroad construction and maintenance.

Chief engineers of railroad companies have therefore come to realize the importance of having a competent bridge engineer as a permanent member of their staff, with duties and responsibilities depending on the number and importance of the bridge structures on the railroad concerned.

It therefore does not appear out of place to point out some of the advantageous results of this arrangement, even when the work concerned can claim no more pretentious classification than ordinary plate-girder construction.

In the first place, whether the structures are fully designed by the railroad company's bridge engineer or not, the main dimensions and general details can be determined, and he can furnish the bridge companies, invited to bid on the work, all the information needed to make an intelligent bid without loss of

time, and with a minimum amount of office work. Sufficient detail will be given to insure that all bidders will figure on the same basis; and if the bridge company is to erect the work, the letter of invitation will clearly indicate how the steel work can be delivered at the site, the character of the stream spanned, the profile of the bottom, and the nature of the false-work where required.

In other words, he will make the proposition so definite that it can be intelligently considered in the bridge company's office promptly and with the least possible allowance for uncertain contingencies. Ability to present the situation in this way enables a railroad company to get bids from the greatest number of responsible bidders and to get the best work for the least money.

If the structures are to replace older ones on an established line, comparative studies and estimates will usually be necessary, involving changes of grade and alignment, the construction of new and the utilization or modification of old masonry, the handling of traffic previous to and during erection; and the knowledge and advice of the bridge engineer will be of prime importance at all stages of the investigation.

If the structures are for an entirely new line, and there are a considerable number of them, the local conditions at the various sites can be studied and classified to secure the greatest possible uniformity of construction and duplication of parts, factors having much weight in the determination of the price which will have to be paid for the work.

For instance, for the majority of small stream crossings where it is impossible to cross at right angles, it is, however, unnecessary to place the masonry in the stream exactly parallel to the current, but one or two angles can be adopted for the skew of piers, one or the other of which will fit the direction of the current accurately enough for practical purposes.

An important practical consideration in this connection also is the desirability of making the abutment ends of such crossings at right angles to the track, regardless of the direction of the current or the skew of the piers. The importance of this point comes from the difficulty to maintain track in good surface at the ends of bridges where one end of several ties rests on a steel girder or masonry, and the other on yielding ballast or earth embankment. By flaring the abutment-wing walls properly, and possibly increasing the length of the bridge slightly,

it is usually possible to make these abutment ends square; and it is well worth doing at considerable cost.

Finally, a railroad company having its own bridge engineer can advantageously take care of the erection of at least all plate-girder spans, simply receiving the steel work on cars at a convenient point on its own lines, and making arrangements for erection with regard solely to its own requirements for safe operation if on an established road, or for construction and tracklaying if on a new line.

The railroad company will then also place the timber, deck and track, and paint the structure after erection with its own forces and at a considerable saving of expense.

In short, the more completely equipped any railroad company is to obtain the best and most substantial structures suited to its needs in the shortest time and at the least cost, and the more completely all operations on or under its own tracks are under its own direct control, the stronger its position both as a public servant and as an earner of dividends.

CHAPTER XII.

ESSENTIALS OF SHORT-SPAN RAILROAD BRIDGES.

*By Mason R. Strong, Consulting Engineer, Erie Railroad.**

THE first essential in bridge engineering is to refrain as much as possible from building bridges; the second, to make the necessary structures of ample capacity to take the water which is coming by. Afterwards the other capacities can be investigated; but even before them comes the type. What is a good kind? A small percentage only of all bridges built have been knocked down. Such a mishap is, however, of not infrequent occurrence. It happens in numerous ways, — by a derailment on the track adjacent to the bridge, and a straight hit on its end post; by the parting of a freight train and the crashing together of the parted sections on the structure; by the dropping from a loaded car of some bulky freight which becomes wedged between the truss and the train; by a straight head-on collision between two trains; by a derailment which might be caused by a broken car axle; by the ice and floods which have of late years carried away so many highway and some railroad bridges, and which will, as forest lands become denuded, increase in distinctiveness; or by collisions of passing boats. All these and other things are among the chronicled enemies of bridges. Not the least is corrosion, be it from atmospheric causes, acids or gases.

And the structure safest against all these vicissitudes is the one of few but stock members which projects above or below the floor but little. Such is the plate girder. Often capable of showing a decreased first cost, it always, when well designed, stands as the best and safest possible structural investment for the carriage of traffic and as insurance against loss in emergencies.

At a certain point a deck-truss bridge could not be made shallow enough, and a through-truss structure was therefore apparently indicated; but a shallow deck-plate girder answers every purpose of the through truss, and stands to-day comparatively invulnerable at a less cost than that of the through span,

* Until recently Engineer of Bridges and Buildings, Erie Railroad.

though its length is over 132 ft. Barring difficulties of transportation, nothing can compare in desirability with such treatment of such spans. At another point where a through-plate girder span over 123 ft. long was built, there occurred a derailment of a freight train which might have tied up the railroad for days had a truss bridge existed. The plate-girder span withstood the assault, and required only ordinary repairs. Again, the arrangement of a city's streets underneath a railroad was such that a through bridge overhead was well-nigh impossible, and also, with the necessary track arrangement, undesirable. An extremely shallow deck-plate girder came to the rescue. It is well, however, to be careful to use as few exceptionally shallow spans as possible unless great leeway for future increases in train loads is made, for, while the girders will no doubt carry the increase with perfect safety, they may then ride poorly by reason of undue deflection; so that if an extremely shallow span is to be designed, it would be wise to design it for a more liberal live load than its neighboring spans of ordinary depth.

Moderately shallow spans, especially moderately shallow main girders in through bridges, are desirable. The girders are less in the way. Their sections are less delicate, if such a word is here permissible. When a little care is used in the design of a single-track span, it may often be found that the girders can be so arranged as to project but little above the track, and thus maintain a good clearance even though the standard distance between track centers is not increased. Their ends are curved on top, and the first top cover plate is carried over the bend. The cars of a derailed train will thus skid over them with a minimum of damage. The appearance of the curved ends is also pleasing. When it becomes necessary to build additional tracks at such a bridge, they will not have to be "spread" apart to maintain the clearance; and the designer may even find that he has presented his client with room for an extra track by his forethought.

In such through spans with girders between the tracks, it is surely permissible to design the "center" or interior girders for but a fraction of the total live load. If three-quarters of the total live load is taken, the stresses in these girders will not become relatively too great when one considers how firmly a center girder in a properly designed bridge is braced and stayed against vibration and motion. The top flange has gussets supporting it on either side, and the bottom has the double floor

and lateral system. Besides, the loads on each track are seldom in exactly the maximum position at the same time. But this is not the thought in such a design; the thought is, as at first stated, that a "center" girder really has more help by reason of its position, and should therefore be called upon to do more work.

Care should be taken, however, in designing under specifications where the allowable stresses are dependent upon the relative intensities of the live and dead stresses, that the full and not the fractional live load is taken in getting the allowable unit stresses. If this is not done, it is apparent there is a double reduction in the capacity of the "center" girder, first, by reason of the lighter live load taken, and, second, because of the consequent exaggerated unit stress.

A double-track bridge with but two girders would be designed for the full live load, for its girders are not rigidly held *on both sides*.

The most vulnerable parts of an old-fashioned girder span, those which show wear and tear first, are the splices of the flanges, the rivets connecting the flange angles to the webs near the ends over or near the bearing plates, and the bracing connections; and, if the girders are deck and spaced too far apart, the web immediately under the top flange angles, where deflection of the wooden floor has tended to bend the web little by little, year after year. Especially is this the case in three-stringer two-track bridges, where the ties butt over the center stringer, — an execrable detail, but one it is possible sometimes to see. With girders spaced thus, this tendency can only be resisted by the best of web stiffeners bearing snugly against the upper flange. Many a through bridge, where the lateral system was connected directly to the floorbeam webs or floorbeam flanges, has given trouble because no efficient method has been provided to take the lateral stresses from the floorbeam to the main girder, the floorbeam web cracking and splitting in consequence. Many shallow through plate-girder bridges have been built with the ties resting directly on the bottom flange angles of the main girders, and some still are so constructed. In these the tendency to bend the webs exists just as in the deck spans mentioned, only, of course, at the bottom instead of the top of the girders; and it is hard to keep the web-stiffener angles on such spans everywhere tightly riveted to the webs, unless all stresses are made moderate. The hitch angles of all

beams should engage with the flange angles. The detail at *H* is a poor one and most suspicious. Wherever it is met with in existing structures it should be the subject of rigid and tireless inspection, and in new construction should never be used at all.

If, then, these old bridges could have had a little more attention given to their splicing, to the riveting together of their component parts near the ends, to their lateral connections, to the proper support of their flanges with web stiffeners against twisting, to the end connections of the beams and stringers of their floors,—their life would have been considerably prolonged.

In the construction of large plate girders it is essential that the webs in the end panels bear on the bottom cover plate (which of course is extended the full length of the girder) and through it on the sole and bed plates. This web plate should be ordered full depth over angles for the end panels and then set so that it may be chipped flush with the face of the bottom flange angles before the cover plate is put on. This is in addition to the ordinary stiffening by bearing stiffeners over the bed plates. It is a feature on which too much insistence cannot be put. An efficient connection between the web and the flanges is also necessary. This is well attained by the use of four flange angles, as shown at *P*. The four heavy angles lessen the number of cover plates,—an important thing, especially in deck spans where ties must be framed over these plates,—and thus shorten the rivets connecting them with the flanges. The splicing of the webs for bending stresses is accomplished by the longitudinal web splice plates shown in the same sketch.

The webs of long and heavy girders are often made considerably thinner near the center of the span than at the ends of the girder. This is done for the sake of economy. A more symmetrical girder is obtained by keeping these plates of the same thickness throughout, and counting upon them to help against the bending stresses at the center, splicing for this at their joints.

It is certainly fair to count one-eighth of the web area as acting thus; but it is wise in making up the parabola for determining the lengths of the cover plates, to place this one-eighth in the top of the parabola, so that its effect may be to exaggerate these lengths instead of to diminish them. Liberal cover-plate lengths are good things. Especially should a plate never be stopped just short of a web splice; carry it over this weak-

ened part of the girder. In a through bridge, too, do not stop a cover plate just short of a panel point.

After a cover plate has reached its theoretical end there should be added enough length to take, with liberal spacing, the rivets indicated by the quotient of the value in single shear of one rivet divided into the area of the plate multiplied by the allowable flange stress ; and it is recommended that the lengths of top and bottom cover plates be alike, and the areas of the flanges too.

After a good masonry foundation is secured, the problem is to distribute the weight uniformly over it. Great care must be taken that deflection of the girders may not bring the bearing on the edge or heel of the bed plate. This has caused the general introduction of pin-bearing pedestals. For spans of moderate length these are oftenest made of plates and angles with a large pin, all bearing surfaces being carefully planed, and all holes being truly bored. For the larger spans — those above 80 ft., or those which for some special reason sustain very heavy loads — cast-steel pedestals are best ; and when the expense of the special workmanship on a built-up pedestal is considered, their cost is not great. They are clean and neat, and will resist all sorts of eccentric stresses in the event of any adverse vicissitudes to the masonry. Pedestals should be designed so as to give enough room between the bottom of the girder and the masonry bridgeseat to get in heavy jacks. This is a convenience if ever the span is to be shifted or raised in ballasting the track.

The expansion ends of the longer, heavier spans rest upon rollers. It is important that these be kept truly in line. This is well accomplished by using center guide straps riveted to the bed and sole plates, the rollers being turned down to receive them, and also having the ordinary guide bars at each end.

The position of the expansion end is important. It should be at the up-grade end of the bridge, where there is a grade. The tendency of the mass down the grade then exerts a push against the abutment, helping it instead of pulling upon it, as it would were it fast at the up-grade end. Where no grade exists there is sometimes a local tendency for the track always to creep. Consider this in placing the free end.

It is sometimes wrong to put the expansion ends all on one abutment. This occurs in a bridge with a violent skew, as shown in diagram. To have put the rollers all at one end of

this bridge would have caused greater distortion in it from temperature stresses than to arrange it as shown.

Where two spans are adjacent on a pier, it is a very common practice to put a continuous bed plate under the pedestals of each, connecting them. This is very wrong. If one span is carried away by ice, or any cause, the adjacent span is very liable to be pulled off the pier with it. The writer remembers seeing two large spans thus connected in course of erection. One was completed, but the second was not yet finished when a freshet with ice occurred, almost carrying out its false works. Fortunately the span was saved; but, if it had gone, its mate would also have been unseated, and double loss have been incurred.

Next above the pedestal, or sole plate, comes the first cover plate of the bottom flange, which, as well as the first top cover plate, should always be made to extend the full length of the girder. Above it and bearing on it are the angles and the web. Bearing on the angles are the web stiffeners, heavy enough to take as columns the full end reaction and transmit it to the web, and straight, not crimped.

Fastened to the rear interior end stiffener is the end cross-frame, designed with diagonal angles sufficient to take to the bed plates all the top lateral loads at moderate unit stresses, and with top and bottom horizontal angles to take the horizontal components. If there were three stiffener angles over the pedestal the end cross-frame might be connected to the center stiffener, though it is usually preferable to connect it to the rear one of the three. These frames should be connected to the flanges as well as to the stiffener angles. This is wise in square bridges, but it is imperative in skew bridges, otherwise the sway of the bridge gets to the diagonal angles of the cross-frame only through the bent connection plate at *C*.

In double-track deck bridges with a girder under each rail it is not desirable to put between adjacent spans cross-frames or anything that will take shear from span to span. It results in loose rivets in the connections; and the work is thrown away, unless the connections and the frames themselves are made inordinately heavy. A lateral system between spans in the plane of the top flange is, however, a good thing. In fact, too rigid bracing can hardly be employed.

To get rivets in their connections without crowding, lateral angles are often put with their longest legs horizontal. The

great objection to such a design is that a space sometimes as wide as twelve inches is made on which lodge cinders, brine, and drippings from refrigerator cars, organic matter from dining cars, and moisture, which rapidly destroy the member unless it is constantly cared for. Put the other way up, a great saving in width is effected; and as good a connection can, nevertheless, usually be designed. Big lateral plates should have good-sized drainage holes, and an effort should everywhere be made

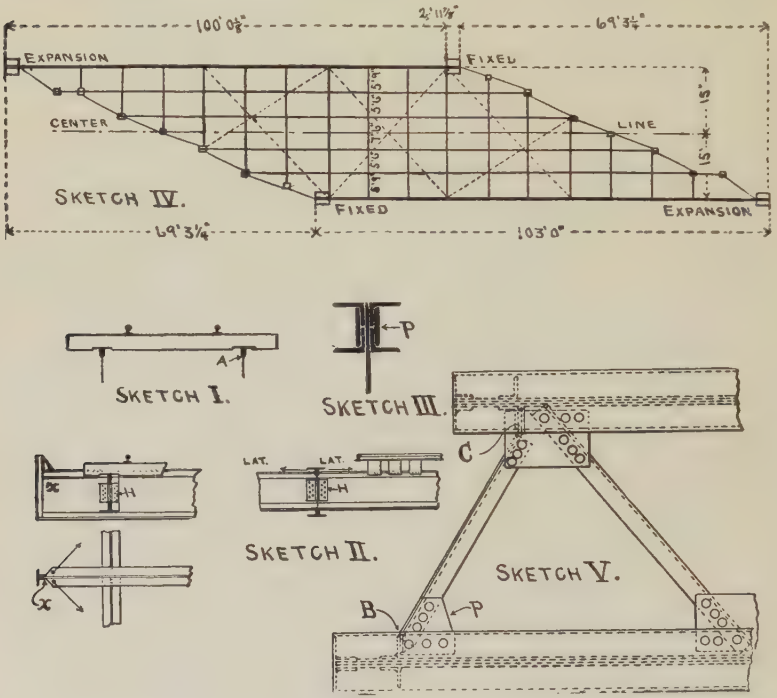


Fig. 140–142.

to avoid lodging-places for dirt and moisture. Closest attention to the means for getting rid of standing water on flat surfaces always pays.

One of the grand reasons why the transverse floor-beams of through bridges should always have cover plates, is that without at least one top plate (extending full length) these foreign substances lie directly on the flange angles and attack them and the web between them, doing irreparable damage. A floorbeam without cover plates may be called "cheap." Soon it will be

"nasty" too. When stringers are placed under, or nearly under, the rails they are not in the line of the drip from passing cars, and may with more propriety be designed without cover plates.

For bridges on curves the best method of obtaining the elevation of rail is often a matter of discussion. In bridges of moderate lengths this may be satisfactorily accomplished by tilting the span somewhat and putting the remaining difference into the gaining of the ties. For the larger spans it is recommended to keep the structure plumb and put the elevation into the ties. In a long bridge where the girders or stringers are in a continuous line, every tie must of course be framed differently, otherwise there will be a hump in the track where it has run up the slant floor and then down again by an amount equal to the middle ordinate of the curve in the length of the bridge.

Plate girders are of course extremely easy to proportion. Armed with a moment and shear curve one can find the stresses from an engine loading as readily as from a uniform load, and from them quickly get the allowable unit stresses. Where a particular engine and specification are constantly used, tables prepared in the following forms are great labor-savers. The columns are such that each page contains the data for two feet of length, if the figures are given for spans varying by three inches in length.

In the tables for through and deck beams, the item "number of rivets, bearing in 1-in. web," gives at once the rivets necessary to take the end reaction; thus, if the web thickness be chosen as one-half inch, the number of rivets is twice that shown. So also the last item, "square inches (net) in flange, for one foot deep," immediately determines the section of the flange. If the girder is made four feet deep, the area of the flange will be one-quarter of that shown in the table for one foot. This is convenient in investigating existing girders as well as designing new.

The sheets are printed on bond paper (without a water mark) with an ordinary printing-press, and the data is written in the blank spaces with black drawing-ink. Good blueprints can then be made, and the whole set bound for ready reference.

The shears for through bridges are the end reactions for spans with end floorbeams. It would, of course, be impossible to show the actual web shear in the end panel, without the panel length being given.

A table of areas, net and gross, of angles and also of plates,

TABLE 30. — DECK (OR THROUGH).

C. to C. of Bearings									
Shear, live									
Shear, dead									
Shear, total									
Web (square inches)									
Number of rivets, bearing in 1-in. web									
Number of rivets, single shear									
Masonry bearing, sq. inches									
Moment, live, in foot-pounds									
Moment, dead									
Moment, total									
Allowable stress									
Sq. ins. (net) in flange for 1 in. deep									

NOTE. — Data is for one rail, stresses are given in thousands of pounds.

TABLE 31. — FLOORBEAMS.

Panel Length.									
Concentration, live									
Concentration, dead . . .									
Concentration, total . . .									
Web (square inches) . .									
Number of rivets, bearing in 1-in. web									
Number of rivets, single shear									

NOTE. — Data is for one rail, stresses are given in thousands of pounds.

TABLE 32. — 14-INCH PLATES.

Flanges with 2 L's 6 × 4 × $\frac{5}{8}$ in.

Thickness.			Area of One Plate Only.	Number of Plates.			
Size.	Decimal.			One.	Two.	Three.	Four.
in.							
$\frac{5}{16}$	.3125	Gross	4.3750	16.0938	20.4688	24.8438	29.2188
		Net	3.7500	12.9688	16.7188	20.4688	24.2188
$\frac{3}{8}$	.375	Gross	5.2500	16.9688	22.2188	27.4688	32.7188
		Net	4.5000	13.7188	18.2188	22.7188	27.2188
$\frac{7}{16}$	.4375	Gross	6.1250	17.8438	23.9688	30.0938	36.2188
		Net	5.2500	14.4688	19.7188	24.9688	30.2188
$\frac{1}{2}$	.5	Gross	7.0000	18.7188	25.7188	32.7188	39.7188
		Net	6.0000	15.2188	21.2188	27.2188	33.2188
$\frac{9}{16}$	.5625	Gross	7.8750	19.5938	27.4688	35.3438	43.2188
		Net	6.7500	15.9688	22.7188	29.4688	36.2188
$\frac{5}{8}$	.625	Gross	8.7500	20.4688	29.2188	37.9688	46.7188
		Net	7.5000	16.7188	24.2188	31.7188	39.2188

Whole angle, two holes out for net.

TABLE 33. — TABLE OF GROSS AND NET SECTIONS OF PLATES FROM 12 IN. TO 18 IN. WIDE, AND $\frac{5}{16}$ IN. TO $\frac{5}{8}$ IN. THICK INCLUSIVE.

$\frac{7}{8}$-in. Rivets, 2 Holes Out.			12-in.	13-in.	14-in.	15-in.	16-in.	18-in.
Size.	Decimal.							
in.								
$\frac{5}{16}$	.3125	Gross	3.7500	4.0625	4.3750	4.6875	5.0000	5.6250
		Net	3.1250	3.4375	3.7500	4.0625	4.3750	5.0000
$\frac{3}{8}$	.375	Gross	4.5000	4.8750	5.2500	5.6250	6.0000	6.7500
		Net	3.7500	4.1250	4.5000	4.8750	5.2500	6.0000
$\frac{7}{16}$	.4375	Gross	5.2500	5.6875	6.1250	6.5625	7.0000	7.8750
		Net	4.3750	4.8125	5.2500	5.6875	6.1250	7.0000
$\frac{1}{2}$	.5	Gross	6.0000	6.5000	7.0000	7.5000	8.0000	9.0000
		Net	5.0000	5.5000	6.0000	6.5000	7.0000	8.0000
$\frac{9}{16}$	.5625	Gross	6.7500	7.3125	7.8750	8.4375	9.0000	10.1250
		Net	5.6250	6.1875	6.7500	7.3125	7.8750	9.0000
$\frac{5}{8}$	.625	Gross	7.5000	8.1250	8.7500	9.3750	10.0000	11.2500
		Net	6.2500	6.8750	7.5000	8.1250	8.7500	10.0000

will of course also form a part of the designer's equipment; and, where a certain size of flange angles is much used as here shown, it is a great convenience. It is given on separate sheets for plates of different widths; and the repetition on each sheet of the area of the plate, with the decimal equivalent of its thickness, is worth the small initial trouble it takes to put it there.

One grade of steel with an ultimate strength of from 56,000 lb. to 64,000 lb. per square inch is recommended. Every piece including the stiffener angles should come up to the full standard

of excellence, and all should be inspected in the mill as well as in the shop.

Where bridges are to be erected immediately after shipment, no painting is better than good boiled linseed oil. If there is a prospect of their standing for some time after shipment, paint should be used either as a first coat or over the oil immediately after unloading. Field-riveting is accomplished under most favorable circumstances with an oil coat only, and it is a fact that the presence of some of the heavier paints in the field-rivet holes make tight rivets somewhat harder to get.

One color only, of some dark shade, is better than a combination of colors; for in the painting, as well as in all other parts of the design, straightforward simplicity is most desirable.

And the first essential in the art of bridge engineering is to refrain as much as possible from building bridges. Much money may at times be saved, if not in first cost at least in future maintenance, by deviating a stream which crosses a road and returns again; by joining two adjacent streets, as a "Y," to span the obstruction, be it railroad or stream, on one structure; or in other simple ways which when sought may suggest themselves in particular instances. Magnificent bridges are being carefully designed and built every day, but the best thing of all for the economical carriage of traffic is the good old earth itself.

CHAPTER XIII.

RAILROAD PLATE GIRDER METHODS AND STANDARDS.

J. E. Greiner, Engineer of Bridges and Buildings, Baltimore and Ohio Railroad.

Office Method of Designing.

We make strain sheets showing the maximum strain, unit strain, area, and sectional make-up of each member, also the rivet pitch in stringers, floorbeams, and plate girders. These strain sheets, with our specifications, are submitted to the contractors for the purpose of obtaining bids. When the contract is awarded, the contractor may immediately proceed to order his material. I have found this method much more satisfactory than by having the bridge companies furnish their own design. In the latter case we frequently have to make changes and corrections, especially in the sections of such members which our experience shows that more section is required than the calculations indicate. By our present method we get just what we want, and all parties bid on precisely the same design.

I understand that good and practical strain sheets require skilled and experienced men in their preparation. We have such men for this purpose. Unless a railroad company has skilled designers it will be better to have the bridge companies submit their designs, and the chances are that the resulting structure will be made better.

Standards.

We have developed standards for deck-plate girders up to 105-ft. span, our object being to indicate to the contractors what we require in the way of details. I do not consider that there are any special features in these designs which are worthy of special mention. We simply endeavor to produce a good, substantial structure, designed as economically as first-class construction will permit. You will note that our lateral bracing is heavier than the usual calculations warrant, but the usual specifications do not cover fully the design of lateral bracing in deck-girder work. There must be provision made for the side lurching of the rolling load, or the girders will have a greater

lateral than vertical deflection. It is only reasonable that bridges designed for Cooper E-50 should have greater lateral stability than those built twenty years ago for E-20. We have been forced to recognize this fact by the action of the older bridges under the heavy power, and have fixed a limit of section in the bracing, which we believe is sufficiently heavy.

It is quite a common practice to consider $\frac{1}{8}$ of the web available as flange area, and then splice the web by side plates near the flanges to take up the horizontal strain, and use vertical splices for the shears. Instead of using horizontal splices in this manner, we use vertical splices with two rows of rivets on each side of joints, and consider these side splices effective for flange area so far as they will go, and make up the balance of the $\frac{1}{8}$ of web by increasing the area of the flange plates slightly.

We use high castings under all girders so as to keep the girders clear of the masonry and the cinders which always accumulate. For spans 80 ft. and over, we use hinged bolsters and rollers.

Considerations Involving Selection of Plate-Girder Spans.

For spans 80 ft. long and under there is really no alternative to the use of plate girders. Of course, riveted trusses could be used for such short spans, but they have no advantage as to economy, and are not so well adapted for the fast and heavy power, and occasional derailments. For lengths between 80 ft. and 100 ft. the question is between a plate or a lattice girder. A lattice girder over 80 ft. long, if in a deck bridge, can be made just as substantial as a plate girder, but there will be little or no saving in cost. Where the length goes to 100 or 105 ft. there is an economy in lattice girders, but on account of the substantial and solid appearance of plate girders, and their good service generally, my practice is to use them up to 100 or 105 ft., wherever conditions will permit their being shipped in one length. When conditions require field-splicing, I prefer lattice girders.

Solid-Floor Bridges.

There are many kinds of ballasted floor designs. I have tried a number of them, but am still looking for something better. The ordinary trough floor, with ballast between the troughs, is a nuisance on account of the noise and difficulty in properly tamping the ties. I-beams covered with buckle plates, or flat

plates, give a better floor than troughs, but this type takes up more depth than troughs. It is not quite so noisy as the trough type. There is another type, which consists of transverse I-beams about 18 in. center, with two plates about 30 in. wide, continuous for the whole length of the bridge, directly under the rails, and to which the rails are clamped. This is not strictly a solid floor, inasmuch as there is no ballast used, and its only advantage is its shallow depth. It is a nuisance so far as noise is concerned.

The latest type of solid floor is composed of I-beams, spaced about 18 in. center to center, and filled with concrete, which completely incases the beams. Ballast is placed on top of the concrete. I believe this the best type of solid floor that I have tried, although the tests are not of sufficient duration to determine whether or not the concrete will adhere permanently to the I-beams. We have had some in use for over a year, and they are still intact and free from the ponderous noise of the other types.

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